FINAL DRAFT REPORT

IN-DELTA STORAGE PROGRAM
RISK ANALYSIS

Prepared for
United States Department of the Interior
Bureau of Reclamation
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And

California Department of Water Resources

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1.1 BACKGROUND

CALFED Record of Decision (ROD) identified In-Delta surface storage projects for further investigations with project-specific studies. The project-specific review is focused on the necessary feasibility studies and environmental review for implementing or proceeding with the In-Delta Storage Project (Project). Based on Reclamation and DWR completed reconnaissance-level studies, information summarized in a report titled, Executive Summary Delta Wetlands Project, USBR/DWR In-Delta Storage Investigations, a decision was made in October 2000 to seek Federal authorization for a feasibility study of alternatives. DWR and Reclamation, in conjunction with CALFED agencies are reviewing the overall feasibility of the Delta Wetlands Project (DWP) for lease or purchase. An alternative modification of the project to meet the State and Federal design standards is also being studied. The ROD has established the following decision points for the In-Delta Storage Program:

- Select project alternative and initiate negotiation with Delta Wetlands owners or other appropriate landowners for acquisition of necessary property by December 2001.
- Develop project plan that addresses local concerns about effects on neighboring lands, and complete any additional needed environmental documentation by July 2002.
- Complete environmental review and documentation, obtain necessary authorization and funding, and begin construction by the end of 2002.

The project plans to store authorized water during winter and spring runoff on two Delta islands, Bacon Island and Webb Tract, and release the water later in the year for beneficial use. The planned reservoir islands are shown on Figure 1-1. The project is fully described in the Draft EIR/EIS prepared by Jones & Stokes Associates (Draft 1995, and Revised Draft 2000).

1.2 PURPOSE, OBJECTIVES, AND SCOPE OF WORK

1.2.1 Purpose

The purpose of the risk analysis is to assess the severity and consequences of failure of the proposed Delta wetlands Project and evaluate its impact on the environment, water quality, reliability of supplies, facilities and infrastructure, economics, public health and safety, and land use. Changes to the DW project will be proposed based on this evaluation.

1.2.2 Objectives

The objective is to assess the probability of risk of failure of the proposed Delta Wetlands Project. Failure shall be defined as an uncontrolled release of water from the reservoir or into the reservoir from adjacent river channels, either from a embankment or appurtenant structure. Risk shall be quantified in terms of exceedance and consequences of failure scenarios will be described qualitatively. Findings will be documented in a report.

1.2.3 Scope of Work/Task Descriptions

The scope of work/task descriptions are from the statement of work for the Delivery Order and are presented below:

Task 1.0 - Operational Risk Analysis

Assumptions and General Approach:

- Consider potential failure modes and their associated probabilities for operational conditions excluding seismic events.

Task Activities:

- Review available aquifer, groundwater, and soil data. Data will likely include monitoring well data, tidal gage readings in the delta, previous aquifer tests, and field and laboratory soils test results. Review proposed project reservoir operations and seepage control measures. Review relevant publications related to the delta levee seepage and stability evaluation.
- Develop ranges of material strength parameters of the various embankment and foundation soils based on the above data review and empirical correlation relationships.
- Estimate seepage conditions and conduct static stability analyses for the proposed Delta Wetlands Project. The analysis will include an assessment of seepage and stability under the following three conditions: (1) immediately after construction is completed: (2) after long term use over the life of the project; and (3) under a rapid draw down scenario. Yield accelerations shall be calculated for the long-term conditions for the most critical slip surfaces for each cross-section. These values will be used later in the simplified deformation analyses of Task 2.
- Compare the range of estimated seepage conditions and the factors of safety from the static slope stability analyses to other Delta islands operating under similar conditions; i.e., Clifton Court Forebay.
- Estimate the potential for piping in the embankment and foundation layers. Estimate the probability of uncontrolled release resulting from piping and slope stability.

Task 2.0 - Seismic Risk Analysis

Assumptions and General Approach:

- Use CALFED PSHA findings as the basis of the base motions for this study. Assume that the ground motions within the center of an island will be used for that island levees.
- Use CALFED Seismic vulnerability sub-team work on levees fragility and vulnerability results to build upon for the specific two islands.
- Define failure criterion using such failure modes as: permanent deformation, cracking piping, and others leading to uncontrolled reservoir release.
- Develop confidence levels around the best estimate profile, material, and stratigraphic parameters to assess the uncertainty and potential variations of these parameters.

Task Activities

- Evaluate material properties of embankments and foundations for dynamic analyses. These properties will include dynamic parameters (such as Gmax, G/Gmax, Vs) of the various strata for comparison with those used in the CALFED studies.

- Use recent CALFED seismic vulnerability study to develop a range of earthquake events (up to four) and their associated probability of occurrence. Based on the work already developed by the seismic vulnerability sub-team of CALFED, we will develop ground motions at the site relating stiff soil site peak ground accelerations to probability of exceedance.
- Use the relationships between stiff soil accelerations, average peak acceleration and levee displacements from the CALFED studies, together with the estimated yield accelerated (item 1.3) to estimate the permanent deformations induced by earthquake shaking.
- Estimate seismic vulnerability of both reservoir islands identifying the probabilities of embankment failures based on the pre-defined failure criteria and the results of the CALFED studies. Compare the seismic vulnerability of the Delta Wetlands Project islands to Clifton Court Forebay.

Task 3.0 - Flood Risk Analysis

Assumptions

- Assume the probabilities of overtopping from inside the reservoir islands, under various wind setup and wave run-ups, are very small.

Task Activities:

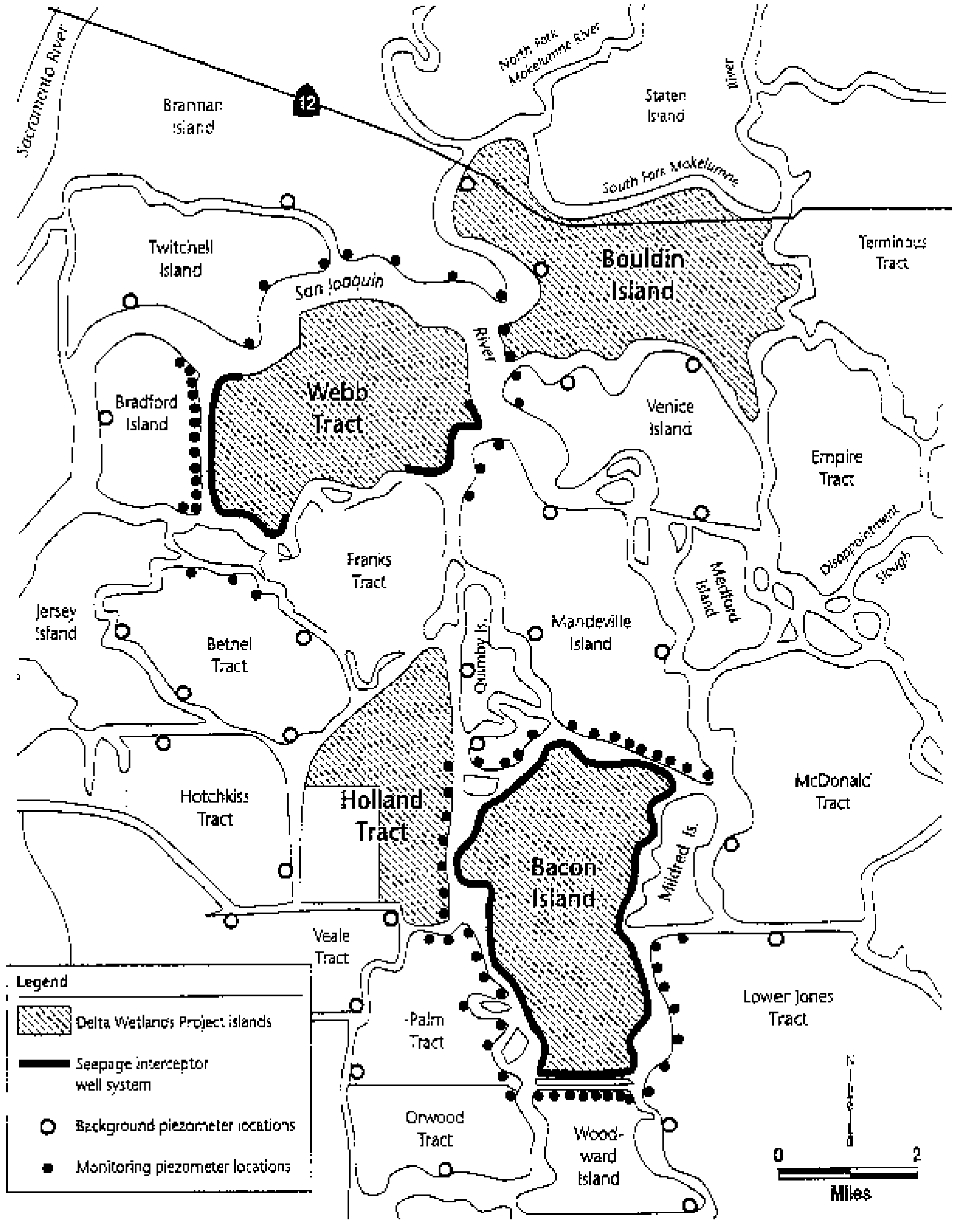
- Review the historical flood events in the Delta from CALFED and previous studies, including information being collected and compiled by DWR Floodplain Management program and the Corps of Engineers Comprehensive Study. Review studies of predicted flood events in the Delta and their associated probabilities, including preliminary and final computer model runs with the input and output data for evaluations being completed by DWR and Corps of Engineers.
- For the flood events in the Delta channels (100-year event) estimate the probability of embankment breach and inward release based upon information developed in Tasks 3.1.

Task 4.0 - Estimate Consequences of Failures

- Consequences of failure shall be evaluated qualitatively for operational, earthquake and flood failure modes. This task will be conducted in coordination with the EIR consultant.
- Estimate consequences of failure qualitatively (e.g., high, medium, and low impact) for each scenario event and each operational scenario.
  - Environmental impact
  - Impact to water quality and reliability
  - Impact to facilities and infrastructure
  - Economic impact of service interruption
  - Impact to public health and safety
  - Impact to land uses

1.3 AUTHORIZATION

The work for this study was performed under Bureau of Reclamation Delivery Order 01A620210H for the In-Delta Storage Pre-Feasibility Study: Requisition Number 01203000105, Contract 01CS20210H for Water Resources Engineering and Environmental Planning Services.



(Ref: Jones & Stokes, 1995)

RESERVOIR LOCATION PLAN
Figure 1-1

2.1 GENERAL

The operational risk analysis involves evaluation of Delta levee seepage and slope stability, potential failure modes and their associated probabilities for operational conditions excluding seismic events.

2.1.1 Slope Stability Analysis

The main objective of the stability analysis is to evaluate the risk of failure for the constructed reservoirs at Webb Tract and Bacon Island. We conducted sensitivity analyses regarding the angle of the slopes on the slough side of the levees, the depth of adjacent sloughs, and possible variability in the average strength of the weakest material (peat). This was done to attempt to quantify how the uncertainty or variability in such parameters influences the overall risk. As part of this assessment, we performed the tasks listed below:

- Reviewed soil engineering parameters and analysis results used in previous applicable studies, or in other studies for similar materials (e.g. peat);
- Defined subsurface soil, reservoir water levels, slough water levels, and phreatic surface within the embankment conditions for slope stability analysis;
- Conducted static slope stability analyses for representative sections of the embankments of the two proposed reservoirs, for various conditions including long-term normal operation and sudden drawdown of the reservoir;
- Performed pseudo-static analyses to define the yield acceleration. This task involved the evaluation of post-liquefaction residual strength of liquefiable sands, stability analyses for the proposed reservoir embankments and estimation of yield accelerations. The yield acceleration were later used in Section 3.0 for the seismic stability analyses.
- Evaluated the Potential for piping failure

The pseudo-static analyses defined the yield acceleration to be used in the assessment of the seismic risk (see Section 3.0). The yield acceleration (K_y) is the horizontal load coefficient, expressed as percent of gravity, which results in a factor of safety of 1.0 for the analysis section considered.

The slope stability analyses were performed using the computer program UTEXAS3 (Wright, 1992). The program uses limit equilibrium analysis based on Spencer's procedure. In Spencer's procedure, side forces acting on a slice interface are assumed to have the same inclination. All requirements for static equilibrium are satisfied. The trial-and-error solution coded in the program involves successive assumptions for the factor of safety and side force inclination until both force and moment equilibrium conditions are satisfied. UTEXAS3 was used to compute factors of safety using either circular or general shaped, noncircular shear surfaces.

UTEXAS3 can perform two-stage and three-stage computations to simulate rapid undrained loading following a period of consolidation of the soil. The two-stage procedure requires the input of both the effective (S-envelope) and total (R-envelope) strength envelopes for cohesive materials such as peat. The sudden drawdown condition was analyzed using the three-stage method described in the user's manual for UTEXAS3. The three-stage method requires as input both the effective and total strength envelopes for peat, and the effective strength envelope for sand.

Liquefaction potential and post-liquefaction residual shear strengths were developed to support the calculation of the yield accelerations as indicated earlier. The potential for liquefaction was estimated based on the mean corrected “clean sand” blowcount $N_1(60)_{cs}$ for a layer identified as prone to liquefy in our review of the subsurface conditions along the perimeter of the islands. Where liquefaction was considered to be likely, the residual undrained shear strength (S_r) of a “liquefied” zone was taken in the lower half of the estimates provided by the upper and lower bound relationships between S_r and $N_1(60)_{cs}$, published by Seed and Harder (1990).

2.1.2 Seepage Evaluation

In the EIR/EIS, an interceptor well system has been proposed to mitigate seepage impacts on neighboring islands as a result of filling the proposed reservoirs. We used the results of previous studies conducted by URS (2000) to briefly review the risk associated with seepage and underseepage, and the potential for piping that could affect adjacent islands. Because the seepage analysis and associated failure modes were already evaluated for the with- and without-project in our previous study (URS, 2000), we have used the findings from that study to address the potential risk due to seepage.

2.2 DATA REVIEW

Various previous geotechnical and environmental studies include information related to the Delta levees and their performance. The reports that were most useful to this data review include: (1) a preliminary geotechnical investigation by HLA (1989); (2) the revised draft EIR/EIS prepared by Jones & Stokes (1995, 2000); (3) an adjunct draft geotechnical report prepared by URS (2000); and (4) USBR’s Status Report for the Delta Wetlands Project (2001).

We reviewed the geotechnical data, assumptions and results contained in the above reports. These reports describe subsurface soil conditions encountered during various field and laboratory investigations at the two study islands and other locations. Previous field investigations included drilling and standard penetration testing (SPT), disturbed or undisturbed sampling, and cone penetration testing (CPT). Previous laboratory testing programs included determination of physical and strength properties. No new field or laboratory work was performed for this study.

For Bacon Island, the field data compiled by URS in 2000 include information gathered from eight borings with SPT, 21 CPT’s, and four monitoring wells. The August 2001 USBR Status Report includes the logs of eight new CPT’s (BI-CPT-01-1 to BI-CPT-01-8). DWR Project Geology provided information on five new SPT borings (BI-SPT1-2001 to BI-SPT5-2001) conducted during September 2001. We found that the 2001 subsurface data are generally consistent with the data previously collected. They confirmed the previously developed information.

For the Webb Tract, the field data compiled by URS in 2000 include information gathered from seven borings with SPT, 26 CPT’s, and four monitoring wells. The August 2001 USBR Status Report presents the logs of eight additional CPT’s (WT-CPT-01-1 to WT-CPT-01-8). DWR Project Geology provided information on four new SPT borings (WT-SPT1-2001 to WT-SPT4-2001) conducted during September 2001. Some of the 2001 data were useful to describe subsurface conditions along the north and northeast portions of the island perimeter, as the previous reports included no information pertaining to those locations.

The SPT boring and CPT logs were used to develop longitudinal profiles of the subsurface conditions along the perimeter of both islands. At both sites, the interior of the proposed reservoirs and the existing levees are underlain by a top layer of peat (PT), followed in depth by successive layers of silty sand (SM), stiff silty clay (CL), and sand.

Because the island and levee ground surface has been subsiding over decades, the existing levees have been raised periodically. The levees are typically built of about 10 feet of sandy to clayey fill, placed on a mixture of clayey peat and peat fill that overlies the natural peat layer. The peat is fibrous, soft, and highly compressible. Its thickness ranges from 15 to 40 feet at these islands. Occasionally, up to 15 feet of fat clay (OH) is encountered between the peat and the silty sand layer.

At Bacon Island, the silty sand underlying the peat is dense to very dense and forms a layer 30 to 40 feet thick. Uncorrected blowcounts (N) in the top of that layer typically range from 13 to greater than 50. Only three N values lower than 5 were found, at the top of the layer, in new boring logs presented in the DW Project status report (August 2001). Other SPT boring data reviewed as part of this study did not show blow counts less than 13. The lowest blowcounts may represent the transition between the sand and peat.

Overall, localized liquefaction of either a very thin layer or isolated pockets of sand, immediately below the peat, could occur under earthquake shaking similar to some of the earthquake scenarios considered in this study. However, based on the data reviewed, and discounting blowcounts greater than 50 but including the three aforementioned low values, the average uncorrected penetration resistance of the upper sands at Bacon Island is N=25. Based on such value, we concluded that the post-liquefaction undrained residual shear strength S_r of most of the upper sands would be higher than the strength of the peat. Therefore, the overall stability of the slopes primarily would be affected more by the low strength of the peat than by the average residual strength of the upper sands. Liquefaction should not control the overall stability of the embankment at Bacon Island.

At Webb Tract, the silty sand thickness ranges from about 30 to 50 feet. A significant difference with the sand layer encountered at Bacon Island is that the upper 3 to 7 feet of the sand layer below Webb Tract appears to be loose and, therefore, potentially liquefiable. For the loose zone of the sand layer, we corrected the low blow counts reported on available field logs. The corrected average penetration resistance of the upper sand (equivalent clean sand) $N_1(60)_{cs}$ was calculated to be 8. Therefore, we conducted post-liquefaction static slope stability analyses for the characteristic embankment sections of the Webb Tract site, based on the undrained residual shear strength for materials assumed to be liquefied.

2.3 ANALYSIS PARAMETERS

2.3.1 Levee and Embankment Geometry

In the text of Sections 2.0 and 3.0, “section” refers to a portion of the levee bounded by two survey stations (e.g., the section of levee located between Station 100+00 and 300+00). “Cross-section” designates the geometry and subsurface profile characterizing any particular “section”. For analysis and risk evaluation purposes, various sections representative of the existing levees were identified along the perimeter of Bacon Island and the Webb Tract. Differentiation between levee sections was based on the average cross-section describing the conditions encountered along the referenced section.

The typical subsurface profile of the representative sections of the levees was defined from available boring and cone penetration testing (CPT) logs. We also used data collected in previous investigations. The thickness of the peat layer encountered within each section was used as the primary element to differentiate between various sections.

The existing levees will be raised and strengthened, on the island side, to form the embankments impounding the proposed reservoirs. The final crest configuration of these embankments was assumed to be at elevation +9 (Delta Wetlands Project), with an initial enlarged crest width (first-stage construction) of 35 feet. Because of the presence of the highly compressible peat below the embankment, consolidation will occur. Such consolidation will be due to the dissipation of excess pore pressures resulting from the new fill surcharge and to secondary settlement caused by long-term creep of the peat.

As the peat consolidates, it will be progressively replaced by the addition of new fill. It is planned that additional fill will be placed to maintain the crest elevation of +9 as settlement occurs (if no settlement acceleration techniques are used). Construction will have to proceed in stages over a period of 4 to 6 years by placing new fill layers above the previous construction stage. After topping off the embankment at +9 plus overbuild, the levee will continue to consolidate. As part of the regular maintenance, the levee will be raised to accommodate the remaining long-term settlement. During the compensation raise, the crest width will be reduced.

Based on information provided on November 6, 2001 by DWR, the inside slope of the reservoir was taken as 5:1 (H to V) at all locations for Bacon Island. The selected inside slope is also 5:1 (H to V) at Webb Tract, except near Station 630+00, where a composite slope will be used. The upper portion of the composite slope, above elevation –3, will be at 3:1 (H to V). The lower portion will be at 10:1 (H to V).

Detailed analysis of the influence of the settlement and construction stages on the stability of the island-side slope of the new embankment is not included in this study. However, an assessment was made to evaluate the influence of the long-term consolidation on the stability of the reservoir embankment slopes (see Section 2.4.1.2).

2.3.1.1 Bacon Island

For Bacon Island, four typical sections were identified, based on the subsurface conditions below the existing levee:

- Section 1 (about 25,300 ft long total): 20 feet of peat, overlying silty sand
- Section 2 (about 21,500 ft long total): 30 feet of peat, overlying silty sand
- Section 3 (about 23,900 ft long total): 15 feet of peat and 15 feet of fat clay overlying silty sand
- Section 4 (about 4,300 ft long total): 40 feet of peat, overlying silty sand

At Bacon Island, the peat is encountered at an average elevation of 0, except at Section 2, where it is encountered at an average elevation of –5. The silty sand layer (SM) underlying the peat typically ranges from medium dense to very dense. This sand layer is followed by a stiff lean clay layer (CL), itself underlain by dense to very dense sand layers (SM, SW and/or SP). Few loose materials seem to exist within the upper sand layer, based on the boring and CPT logs reviewed for this study. Table 2-1 summarizes where the above sections were encountered at Bacon Island.

TABLE 2-1 – ISLAND PERIMETER SUBDIVISION (BACON ISLAND)

Sections	Start Station	End Station	Thickness of Peat and Soft Clay (ft)	Section Length (ft)	Existing Levee Crest EL. (ft)	Reference Number or Source of Information
Section 1	0+00 100+00 350+00 457+50 545+80 716+65	25+00 167+00 404+00 493+75 583+30 750+00	20	2,500 6,700 5,400 3,625 3,750 3,335	+8.0	Borings & CPT Logs Profile Crest El. From URS 2000 analyses
Section 2	25+00 210+00	100+00 350+00	30	7,500 14,000	+8.0	Borings & CPT Logs Profile Crest El. From URS 2000 analyses
Section 3	404+00 493+75 583+30	457+50 545+80 716+65	30	5,350 5,205 13,335	+8.0	Borings & CPT Logs Profile Crest El. From URS 2000 analyses
Section 4	167+00	210+00	40	4,300	+8.0	Borings & CPT Logs Profile Crest El. From URS 2000 analyses

Table 2-2 summarizes the typical geometry of the existing levee at the various sections identified. Available bathymetric survey data were used to assess the average slope geometries described in Table 2-2. In our stability analyses, the new embankment fill was also included.

TABLE 2-2 – TYPICAL EXISTING SECTION GEOMETRY (BACON ISLAND)

Sections	Reservoir (island) Bottom El. (ft) (2)	Island Side Slope Angle (°) (1)	Slough Slope Angle (°) (1)	Steepest Slough Slope Angle (°) (1)	Average Crest Width (ft)	Average Slough Bottom El. (ft) (2)	Lowest Slough Bottom El. (ft) (2)
Section 1	-9	17 (3.3:1)	19 (2.9:1)	30 (1.7H:1)	28	-25	-32
Section 2	-10	15 (3.7:1)	18 (3.1:1)	30 (1.7H:1)	22	-28	-33
Section 3	-9	13 (4.3:1)	20 (2.7:1)	29 (1.8:1)	22	-15	-33
Section 4	-9	16 (3.5:1)	18 (3.1:1)	21 (2.6:1)	28	-30	-33

Notes: (1) Slope angles are measured with respect to horizontal, and expressed as horizontal to vertical
(2) Elevations are based on topographic maps by MBK Engineers (Jan.,96) for Bacon Island and Murray, Burns & Kielen (April,96) for Webb Tract

2.3.1.2 Webb Tract

For the Webb Tract, four sections were identified:

- Section 1 (about 24,000 ft long total): 20 feet of peat, overlying silty sand
- Section 2 (about 36,300 ft long total): 30 feet of peat, overlying silty sand
- Section 3 (about 5,000 ft long total): 40 feet of peat, overlying silty sand
- Section 4 (about 2,900 ft long total): 40 feet of loose materials over silty sand

At Webb Tract, peat is encountered at an average elevation of -5, except for Section 3, where it is encountered at an average elevation of 0. The sand (SM) underlying the peat at Webb Tract significantly differs from that underlying the peat at Bacon Island. The upper 3 to 7 feet of the Webb sand are loose and, therefore, potentially liquefiable. The interpretation of such condition was based on low uncorrected SPT blow counts (4 to 7) and/or simultaneous occurrence of low tip resistance and low friction ratio in the CPT logs. The rest of the sand below the loose sublayer typically ranges from medium dense to very dense. Based on a few deep borings, the two sand sublayers are underlain by a stiff fat clay layer (CH), followed by another dense to very dense silty sand layer.

Table 2-3 describes the four section types that were encountered at Webb Tract. The subsurface conditions at Webb Tract Section 4 differ from those encountered elsewhere around the island. At that section, below the levee fill, approximately 40 feet of materials with relatively low CPT tip penetration resistance (averaging 50 tsf) and low friction ratio were encountered. Section 4 may correspond to a repaired portion of the levee. It was not investigated in this study, because of its limited extent and the lack of information regarding applicable soil parameters.

TABLE 2-3 – ISLAND PERIMETER SUBDIVISION (WEBB TRACT)

Sections	Start Station	End Station	Thickness of Peat and Soft Clay (ft)	Section Length (ft)	Existing Levee Crest EL. (ft)	Reference Number or Source of Information
Section 1	0+00 518+00 614+20	196+75 537+90 637+50	20	19,675 1,990 2,330	+8.0	Borings & CPT Logs Profile Crest El. from URS 2000 analyses
Section 2	196+75 470+75 537+90 637+50	391+75 518+00 614+20 682+00	30	19,500 4,725 7,630 4,450	+8.0	Borings & CPT Logs Profile Crest El. from URS 2000 analyses USBR DW Project, status rpt 8/3/01
Section 3	391+75	441+75	40	5,000	+8.0	Borings & CPT Logs Profile Crest El. from URS 2000 analyses
Section 4	441+75	470+75	0	2,900	+8.0	Borings & CPT Logs Profile Crest El. from URS 2000 analyses

Table 2-4 summarizes the typical geometry of the existing levees at three of the above sections that were analyzed, as defined from available bathymetric survey. As in the case of the Bacon Island Reservoir, such geometry was modified by representing the new embankment fill in our stability analyses.

TABLE 2-4 – TYPICAL EXISTING SECTION GEOMETRY (WEBB TRACT)

Sections	Reservoir (island) Bottom El. (ft) (2)	Island Side Slope Angle (°) (1)	Slough Slope Angle (°) (1)	Steepest Slough Slope Angle (°) (1)	Averag e Crest Width (ft)	Average Slough Bottom El. (ft) (2)	Lowest Slough Bottom El. (ft) (2)
Section 1	-10	16 (3.5:1)	21 (2.6:1)	35 (1.4:1)	21	-25	-30
Section 2	-12	13 (4.3:1)	19 (2.9:1)	33 (1.5:1)	20	-25	-50
Section 3	-11	14 (4.0:1)	23 (2.4:1)	24 (2.2:1)	20	-27	-41

Notes: (1) Slope angles are measured with respect to horizontal, and expressed as horizontal to vertical
(2) Elevations are based on topographic maps by MBK Engineers (Jan.,96) for Bacon Island and Murray, Burns & Kielen (April,96) for Webb Tract

2.3.2 Material Properties

Physical and strength properties were developed from the information provided by DWR and from our review of previous studies, and are summarized in Table 2-5.

TABLE 2-5 GENERAL ANALYSIS STATIC PROPERTIES

Material Description (Type)	Unit Weight (pcf)		UU strength End of Construct.		CD strength Normal Operation		CU strength Rapid Drawdown		Source of Information
	Moist	Satur'd	Cohes. (psf)	Fric. (°)	Cohes. (psf)	Fric. (°)	Cohes. (psf)	Fric. (°)	
Type 1 New Fill for Embankment	110	120	0	30	0	30	0	30	[1]
Type 2 Existing Fill SAND	110	110	0	30	0	30	0	30	[2] [1] Est'd
Type 3 (1) Existing Fill SAND MIX.	110	110	0	30	0	30	0	30	[2] [1]
Type 4 PEAT [under levee]	70 (3)	70	300 (2)	0	50	28	100	15	[1] Est'd
Type 5 PEAT [free surface]	70	70	250	0	50	26	100	15	[2] [1]
Type 6 FAT CLAY [below peat]	N/A	85	250	0	0	25	100	30	[1]
Type 7 SAND [below peat or clay]	N/A	125	0	36	0	36	0	36	[1] Est'd

Notes: (1) "SAND MIX" designates: SAND mixed with CLAY and PEAT
(2) UU cohesion of peat under levee: 50-1500 psf per [1], 135 psf per [2], 100-300 psf per [3]. Used in this table 300 psf average cohesion, because strength should be slightly higher than below free surface.
(3) Used same weight as saturated for moist weight as was done for material Type 5

N/A designates: not applicable
Est'd designates: estimated value, when specific reference is not available

Sources of Information:
[1] Draft Table 3, DWR/USBR, dated November 5, 2001
[2] URS Report to Jones & Stokes Associates, dated March 31, 2000
[3] Harding-Lawson 1989 study

For the upper portion of the sand layer (Material Type 7) at Webb Tract, the post-liquefaction undrained residual strength was taken as 200 psf, based on the average estimated corrected penetration resistance (SPT).

2.4 ANALYSIS CRITERIA

2.4.1 Analysis Cases

The factors of safety of both slopes were assessed, because critical conditions may arise either on the slopes facing the slough side or the reservoir island side. The following analysis conditions were evaluated.

2.4.1.1 End-of-Construction

The end-of-construction scenario is the condition occurring immediately after placement of new fill on the reservoir island side of the levee. Fill is placed in thin layers and compacted. Immediately after fill placement, relatively impervious materials such as peat and clay in the levee and foundation will not have had sufficient time to dissipate construction-induced excess pore pressures. Hence, at the end of construction, undrained shear strengths are normally used to characterize the cohesive soils of the levee and foundation.

2.4.1.2 Long-term Operation

The analysis of long-term levee stability involves the post-construction conditions when strength gain has occurred, and normal operation of the reservoir is in place. Two combinations of water levels (high reservoir and low slough water, and vise-versa) on the island and slough sides were selected to produce the most critical load cases that could be encountered during such operation.

As discussed in Section 2.3.1, the material properties and resulting factors of safety will change as a function of time as the peat consolidates and new fill is placed to keep the embankment crest to its design elevation of +9. Our calculations were performed assuming long-term strength properties, two combinations of reservoir and slough water levels, and the embankment crest at elevation +9 with a first stage width of 35 feet. The influence of consolidation and placement of additional fill was estimated by analyzing one section, representative of average conditions, at Bacon Island. It showed less than 10 percent change in computed factors of safety. The factors of safety tabulated in our calculations are within this range of accuracy.

2.4.1.3 Sudden Drawdown

The sudden drawdown case affects the reservoir-side slope when the reservoir water level drops rapidly. Such condition may result from emergency drainage of the reservoir.

Because the drop in reservoir level can occur at a relatively rapid rate, the peat and other fine-grained soils would not have enough time to drain, and undrained strengths are used in the analysis. Therefore, the phreatic surface within the embankment section would not have sufficient time to drop. The extent of the saturated materials within the embankment section would remain unchanged and would be the same as the normal operating condition.

2.4.1.4 Pseudo-Static Analysis

Pseudo static analysis is used to estimate the yield acceleration (K_y). The use of the calculated yield acceleration to estimate earthquake-induced deformation of the levees systems is discussed in Section 3.0. Water levels on the island and slough sides were selected to produce critical cases.

As discussed in Sections 2.3.1.1 and 2.3.1.2, the potential for liquefaction was considered to be low at Bacon Island, and high at Webb Tract. Liquefaction is the loss of strength of saturated loose granular materials due to volume changes and build-up of excess pore pressures resulting from earthquake shaking. Simplified procedures based on the corrected penetration resistance (Seed and Idriss, 1982; Idriss, 1999) were used to assess the potential for liquefaction. If liquefaction is reached, the stability of the affected soils is governed by the undrained residual strength (Seed and Harder, 1990). A conventional "post-shaking" slope stability analysis can be performed, using such a parameter for the liquefied layer(s).

For Webb Tract, the strength of the liquefiable layer was taken as the undrained residual shear strength to define the yield acceleration. We assumed that the probability of liquefying the upper portion of the sand layer at Webb Tract, should a significant earthquake occur, would be 1.0. We then calculated the post-earthquake factor of safety of the reservoir slopes, using S_r in the liquefied zone. A post-earthquake factor of safety greater than 1.0 indicates that possible instability would be primarily controlled by excessive deformations rather than by liquefaction alone (probability of failure solely caused by liquefaction = 0). A factor of safety less than 1.0 in the post-earthquake static stability analysis indicates that embankment failure primarily due to liquefaction (e.g., flow failure) would be likely. Based on those simplified considerations, the probability of occurrence of failure due to liquefaction is approximated by the probability of occurrence of the earthquake scenario considered, if a statically unstable post-liquefaction condition is predicted.

2.4.2 Reservoir Stages

The reservoirs will operate at various levels during a typical calendar year. Patterns for reservoir levels were developed in other studies (e.g. Hultgren, 1997), as reproduced on Figure 2-1. In a typical year, for a little less than two months (May and June), the reservoirs will be at their maximum operating water level (+4). During the five month period of September through January of the following year, the reservoirs will be at their lowest operating level or will be empty. Figure 2-1 shows an intermediate constant reservoir stage at about –11 in the second week of February to the third week in March. In between these three periods of time, variations of the reservoir level will be approximately linear.

The maximum and minimum levels for the reservoir last for extended periods of time and, therefore, define conditions that correspond to normal operation. Depending on the analysis case considered, the maximum or minimum level may be the most critical for a given slope and analysis case. The most critical of either the maximum or minimum reservoir levels was considered in our analyses.

2.4.3 Slough Water Levels

At each section and case analyzed, we used a combination of reservoir and slough water surface levels that produce critical conditions. A high slough water surface elevation, combined with a low reservoir elevation, is potentially the most critical to the island-side slope. A low slough water surface elevation, combined with a high reservoir elevation, is potentially the most critical to the slough-side slope.

For the analysis of the long-term condition of the island-side slope, we assumed that the water level in the slough could reach peak flood level at least once during the design life of the reservoir. The maximum peak flood elevation corresponding to the 100-year flood condition is +7.2 feet at Bacon Island and +7.0 feet at Webb Tract (see Section 4.0).

The sudden drawdown condition does not represent a “normal” condition. Therefore, it is not necessary to consider the same flood condition that is considered for the long-term condition. For the sudden drawdown analysis case, we used a slough water elevation of +6 feet based on a review of historical gauge data applicable to the two sites. In these data, we noted that the maximum peak flood occurs over a very short time, and hence should not lead to a steady-state condition during the relatively short duration of the sudden drawdown. The selected “sustained” flood elevation of +6 feet conservatively represents a critical condition for this analysis case.

For the stability evaluation of the slough-side slopes slough, we took the water surface level in the slough at an average low tide elevation (elevation -1 feet). This represents a reasonably conservative condition.

The water elevations discussed above are tabulated along with the results of our stability analyses (see Section 2.5).

2.4.4 Seepage Analysis

In geotechnical studies supporting the EIR/EIS prepared by Jones & Stokes, URS (2000) performed seepage studies, using two-dimensional finite element models. These studies evaluated seepage conditions, pumping requirements, and estimated the impact of well or pump failure. For each island, two cross-sections were selected (Bacon Island, Stations 220+00 and 665+00; Webb Tract, Stations 260+00 and 630+00). These correspond to current levee Sections 2 and 3 for Bacon Island and levee Sections 2 and 1 for Webb Tract. The cross sections at each island were selected to represent the “narrowest” and “widest” slough width across the reservoir island and neighboring islands, with the intent to bound the range of actual seepage conditions.

For this pre-feasibility study, we used the results of the URS 2000 study to provide an assessment of seepage. An important difference between this and the previous studies is that a higher reservoir level (elevation +6 feet) was considered in the 2000 study. The reservoir elevation considered in this study is +4 feet. Hence, seepage rates and required pumping would decrease in proportion to the maximum head in the reservoir, and the risks associated with seepage and potential piping should be proportionately reduced.

The following findings are taken from the URS report. Therefore, and as previously stated, they apply to a reservoir level (+6) that is more demanding than presently considered for the project (+4). Yet, they should provide reasonable, but conservative assessment of seepage and associated risks of piping.

2.4.4.1 Seepage Evaluation

In 2000, existing conditions were first used to calibrate the numerical models against observed existing conditions. This included levee and subsurface conditions, and existing monitoring well data. Groundwater conditions within the project islands were established from surface water levels in the drainage ditches. Water levels recorded in nearby gauging stations within the Delta were used to define the water levels in the surrounding sloughs. Soil permeabilities were first estimated from empirical correlations based on grain distribution, and then adjusted until conditions similar to the baseline case were matched.

The seepage models were then modified to include each embankment and reservoir, and assess the impact on neighboring islands as a result of reservoir filling. The analyses focused on evaluating the impacts of reservoir filling and modified levee configuration on changes in hydraulic head, exit gradients, flow rates, and groundwater level in the neighboring islands. Based on such computed changes, pumping rate and well spacing requirements were parametrically varied until the baseline conditions "without-project" were restored for a range of assumed permeability values based on available data. Below is the summary of our previous findings.

- *If seepage mitigation measures are not implemented, the proposed reservoir islands will have undesirable seepage flooding effects on adjacent islands.*
- *Seepage control by interceptor wells placed on the levees of the reservoir islands, as proposed by Delta Wetlands (DW), appears effective to control undesirable seepage effects. Required well spacing and pumping rates appear to be reasonable and manageable.*
- *A system of checking well performance and maintenance needs to be developed and implemented. Proper documentation shall be provided to identify wells requiring excessive maintenance and potential adverse de-silting of the aquifer.*
- *During construction, a minimum of 800 to 1,000 feet offset form the levee toe should be maintained for the location of borrow sites. With such offset, there should be no discernible effects of borrow areas on seepage.*
- *Sensitivity analyses concluded that the permeability of the channel silt and aquifer have a significant impact on the seepage conditions and required pumping volume, while reasonable variation of the average peat thickness has little effect.*

2.4.4.2 Seepage Mitigation and Monitoring

In order to reduce the risk of adverse seepage impacts to neighboring islands, DW proposed to install a seepage monitoring system after the implementation of the project. DW also proposed significance standards to evaluate when the review of seepage monitoring data should trigger initiation of seepage control measures. URS (2000) reviewed such concepts and evaluated their adequacy, taking into account historic water level data.

Data collected over the past 10 years from existing monitoring wells were used as a "baseline" to develop the proposed significance criteria. DW plans to install a network of "monitoring" wells (piezometers) in the neighboring islands to monitor seepage data during the project implementation. In addition, "background" wells (far from the reservoir islands) would be used as future baseline reference. During filling and storage stages, data from monitoring wells on neighboring islands will be compared to the historical and background data. The comparison with historical data will establish whether piezometric levels are affected by reservoir filling and storage. The comparison with the background data will be used to assess whether deviations from historic data are observed throughout the Delta, or only at the reservoir sites. Below is a summary of the evaluation and findings of this phase of our previous study.

- *Monitoring and maintaining compliance with significance criteria are essential, and must be carefully adopted and implemented.*
- *Use of a combination of seepage monitoring wells and background wells, as proposed by DW, appears suitable and reasonable. The number of background wells should be sufficient to provide enough redundancy at each row of monitoring wells.*

- *Well readings by means of automatic data acquisition is appropriate and necessary for on-time response, should it be needed.*
- *Significance criteria have been developed by DW, in consultation with others, to be used with the monitoring results to trigger seepage mitigation, as required. Seepage mitigation will first consist of pumping from the interceptor wells. The concept and format of the criteria appear appropriate, but some changes appear desirable.*
- *The significance criteria should be reevaluated and updated periodically.*

2.5 ANALYSIS RESULTS

2.5.1 End-of-Construction

This analysis case was evaluated by URS (2000) for two typical cross-sections at each of the two islands. Two successive analyses, a two-stage analysis (Method 1) and the undrained strength (S_u) analysis (Method 2) were conducted for each cross-section. Analyzed embankment cross-sections were Stations 25+00 and 265+00 at Bacon Island, and Stations 160+00 and 630+00 at Webb Island. Slough water and island groundwater levels were selected to assess a critical condition. New analyses were not conducted as most of the factors of safety computed in our 2000 study were less than 1.0.

In the referenced analyses, factors of safety greater than 1.0 were calculated only when the more conservative composite slope (3H:1V above –3 feet and 10H:1V below elevation –3 feet) was used. Otherwise, the factors of safety ranged from 0.6 to 0.9, for embankment slopes at 5H:1V. In such analyses, all new fill was assumed to be placed at once (one construction season) on the existing levees. The above factors of safety show that this will not be feasible.

The above results indicate the need for careful planning and staging of the construction of the embankment over several seasons (4 to 6 years). These results confirm that building up the embankments too rapidly would result in slope failure. The construction sequence of the fill in a staged fashion can be specified during design and verified during construction. This design requirement may include such criteria as minimum required factor of safety and consolidation strength gain before the next staged layer is placed.

We estimated that the probability of embankment failure with release of water from the adjacent slough into the reservoir area would be significant (say greater than 50 percent), if construction proceeds too rapidly or without staging. However, if the fill is designed to allow careful placement of controlled lifts, allowing proper consolidation and gain of strength, stability during and immediately after construction will be satisfied. The risk should be very small if flatter embankment slopes, as considered for Webb Tract Station 630+00, were systematically used.

2.5.2 Long-Term Normal Operation

For the assumed embankment geometries and subsurface conditions considered, the computed factors of safety for the slough-side slope are about 1.2 at Bacon Island. Factors of safety on the reservoir-side slope are higher, with a lowest computed value of 1.7.

At Webb Tract, the lowest computed factors of safety range from 1.1 (Section 1) to 1.2 (Sections 2 and 3). The lowest reservoir-side slope factor of safety is 1.6.

The results are presented in Tables 2-7 and 2-8 along with water surface elevations assumed on either side of the embankment. Typical embankment geometries and failure surfaces for Section 1 of Bacon Island are presented on Figures 2-2 and 2-3, and on Figures 2-6 and 2-7 for Section 1 of Webb Tract. Failure surfaces for other sections analyzed were not plotted, but are generally similar in location.

2.5.3 Sudden Drawdown

Computed factors of safety for the Bacon Island reservoir range from 0.9 (Sections 2, 3 and 4) to 1.0 (Section 1). For the Webb Tract reservoir, they also range from 0.9 (Sections 2 and 3) to 1.0 (Section 1). These results are based on the conservative assumption that the new fill along the inside perimeter of the embankment would remain fully saturated after the occurrence of sudden drawdown; the results are summarized in Tables 2-7 and 2-8. Typical plots of the critical failure surfaces are presented on Figures 2-5 and 2-9 for Bacon Island and Webb Tract, respectively.

2.5.4 Pseudo-Static Analysis

The pseudo-static analyses were performed to estimate the yield accelerations (K_y) to be used in the seismic risk analysis (See Section 3.0). For Bacon Island, the yield accelerations range from 0.035 (Sections 2 and 3) to 0.05 (Section 1). For Webb Tract, they range from 0.005 at Section 1 to 0.025 at Section 3. The K_y values for Webb Tract are significantly lower than at Bacon Island, because they are based on the consideration that the entire loose sand layer identified at the Webb Tract site has liquefied (see Section 2.5.5). This is a conservative assumption, as excess pore pressure less than 100 percent could be generated in more or less extended areas of the liquefaction-susceptible layer, depending on the amplitudes and duration of the shaking. The results are summarized in Tables 2-7 and 2-8. Typical plots of the critical failure surfaces are presented on Figures 2-4 and 2-8 for Bacon Island and Webb Tract, respectively.

2.5.5 Post-Liquefaction Stability Analysis

As discussed, earthquake-induced liquefaction is more likely at Webb Tract than at Bacon Island, because of the presence of a more continuous and thicker layer of loose silty sand. Hence, we analyzed the post-liquefaction case only for the Webb Tract sections. In that case, the failure surface is non-circular and passes through the “liquefied layer” (assigned the post-liquefaction residual undrained strength). The computed factors of safety are about 1.0 to 1.1 (see Tables 2.7 and 2.8). The development of earthquake-induced excess pore pressures in the embankment materials was not considered, which is potentially unconservative. However, the entire loose sand layer was assumed liquefied, which is conservative. Based on these calculations, the post-earthquake stability of the embankment slopes is marginal.

TABLE 2-7 RESULTS SUMMARY – SLOPE STABILITY ANALYSIS – BACON ISLAND

Section 1

	Water Elevation (ft)				
Condition	**Slough**	**Reservoir**	**Side Slope Considered**	**Ky**	**F.S.**
Long-term	7.2	Empty	Reservoir	-	2.1
Long-term	-1	4	Slough	-	1.2
Seismic, Ky	-1	4	Slough	0.05	
Sudden Drawdown	6	Empty	Reservoir	-	1.0

Section 2

	Water Elevation (ft)				
Condition	**Slough**	**Reservoir**	**Side Slope Considered**	**Ky**	**F.S.**
Long-term	7.2	empty	Reservoir	-	1.7
Long-term	-1	4	Slough	-	1.2
Seismic, Ky	-1	4	Slough	0.035	
Sudden Drawdown	6	empty	Reservoir	-	0.9

Section 3

	Water Elevation (ft)				
Condition	**Slough**	**Reservoir**	**Side Slope Considered**	**Ky**	**F.S.**
Long-term	7.2	empty	Reservoir	-	1.8
Long-term	-1	4	Slough	-	1.2
Seismic, Ky	-1	4	Slough	0.035	
Sudden Drawdown	6	empty	Reservoir	-	0.9

Section 4

	Water Elevation (ft)				
Condition	**Slough**	**Reservoir**	**Side Slope Considered**	**Ky**	**F.S.**
Long-term	7.2	empty	Reservoir	-	1.7
Long-term	-1	4	Slough	-	1.2
Seismic, Ky	-1	4	Slough	0.045	
Sudden Drawdown	6	empty	Reservoir	-	0.9

TABLE 2-8 RESULTS SUMMARY – SLOPE STABILITY ANALYSIS – WEBB TRACT

Section 1

	Water Elevation (ft)				
Condition	**Slough**	**Reservoir**	**Side Slope Considered**	**Ky**	**F.S.**
Long-term	7.0	empty	Reservoir	-	1.8
Long-term	-1	4	Slough	-	1.1
Seismic, Ky	-1	4	Slough (circular)	0.005	
Post-Liquefaction	-1	4	Slough (Noncircular)	-	
Sudden Drawdown	6	empty	Reservoir	-	1.0

Section 2

	Water Elevation (ft)				
Condition	**Slough**	**Reservoir**	**Side Slope Considered**	**Ky**	**F.S.**
Long-term	7.0	empty	Reservoir	-	1.6
Long-term	-1	4	Slough	-	1.2
Seismic, Ky	-1	4	Slough (circular circle)	0.012	
Post-Liquefaction	-1	4	Slough (Noncircular)	-	
Sudden Drawdown	6	empty	Reservoir	-	0.9

Section 3

	Water Elevation (ft)				
Condition	**Slough**	**Reservoir**	**Side Slope Considered**	**Ky**	**F.S.**
Long-term	7.0	empty	Reservoir	-	1.6
Long-term	-1	4	Slough	-	1.2
Seismic, Ky	-1	4	Slough (circular circle)	0.025	
Post-Liquefaction	-1	4	Slough (Noncircular)	0.01	
Sudden Drawdown	6	Empty	Reservoir	-	0.9

2.6 DISCUSSION OF RESULTS

2.6.1 General

Contrary to extreme conditions such as flood or seismic events which can be assigned a probability of occurrence based on historic observations, the risk of failure during operational conditions is difficult to quantify without in-depth studies. Several factors, individually or jointly, reduce the factor of safety under normal operating conditions and, therefore, increase the probability of failure. The most important of these factors are:

- The variability of the in-situ strength parameters, permeabilities and thickness of critical layers, including unknown conditions in areas not covered by the geotechnical exploration
- The variability of the water levels along the embankment perimeter
- The variability of the slough-slide slope geometry
- Slope failure, cracking, or progressive deterioration could lead to breaching the embankment

- Equipment failures, human errors or detrimental actions, and other non-natural events could cause failure of the embankment.

Because of the difficulty to quantify the influence of the above factors, we used qualitative assessments to correlate our stability and seepage analysis results with the probability of failure during normal operation of the reservoirs. If a factor of safety of 1.0 is indicative of slope failure, the probability of such failure is the probability that all possible combinations of the above factors will lead to a factor of safety less than 1.0.

The approach and findings presented below build on the Houston and Duncan study of the Delta failure risk (1978). The work by Houston and Duncan addressed the Delta as a whole and evaluated the probabilities of failure of the levee system without improvements or upgrades. To estimate the risk of the proposed Webb Tract and Bacon Island project, we evaluated the relative changes between the existing and the proposed project levees based on the findings from Houston and Duncan.

2.6.2 Seepage Risk

Based on Section 2.4.4 and the water level (+4) proposed for the reservoir, seepage and piping should not represent a serious risk, if adequate mitigation and monitoring measures are implemented and maintained during the project active life. Seepage and piping would pose a high risk, if such mitigation measures were not considered. We believe that the proposed seepage reduction and monitoring measures will reduce the risk of flooding neighboring island to a low level.

Monitoring will prevent significant seepage increases due to well failures by silting or other cuases, because faulty wells could be identified, and repaired or replaced in a timely fashion. However, a potentially critical seepage condition could result from exceptional events (of very low probability of occurrence). For example, power loss or grid failures have lasted from days to weeks, or even months, in some major historic earthquakes outside the United States. While backup (e.g., diesel operated pumps) is contemplated for the well system, local or distant large earthquakes could cause extended power failures, or even prevent or limit access to the backup pumps for a significant duration of time. Of interest is what is the shortest duration of a pumping interruption required to cause flooding of significance to an adjacent island. Such evaluation, which involves transient seepage analysis (steady-state well pumping rates drop instantaneously to zero), would be appropriate at a more advanced stage of the project.

2.6.3 Slope Stability Risk

The variability and influence of the factors contributing to the overall risk and listed in Section 2.6.1 cannot be readily quantified. Houston and Duncan (1978) evaluated the probabilities of failure of existing levees in the Sacramento-San Joaquin Delta by treating two of these risk factors, the water levels outside the levees and the material strength (effective friction angle) as random variables. They did not consider the influence of the other factors, which could lead to underestimating the risk. They concluded that, in a period of 40 years, there was a probability of failure of 0.02 at Bacon Island and 0.05 at Webb Tract. We used the results of Houston and Duncan's studies to compare the previously computed risk of failure of the existing levees with what could be the risk of failure of the constructed reservoirs, under normal operating condition.

2.6.3.1 Influence of Material Strength

Houston and Duncan found that the peat thickness contributes significantly to the average probability of failure, in any one-year period. This is consistent with our results. For long-term conditions, they assumed an average effective cohesion of 50 psf for peat, which is also the value that we used (see Table 2-5). They selected a mean friction angle of 30 degrees, with a standard deviation of 6 degrees (20 percent of the mean value). This is a higher mean estimate than the 26 degrees (free surface) or 28 degrees (under levee) used in our analyses. Houston and Duncan's factors of safety are higher than would be computed with our strength assumptions. Factors of safety based on our estimated properties should be about 10 to 15 percent lower, for the slope geometries used in 1978. Hence, based on estimated material strengths, the calculated number of failures for the existing levees should be higher than estimated in 1978. For the constructed embankment, slope failures on the slough-side are probably more likely to occur than computed in 1978. Proposed DW slopes is 5:1 on the island side which is not much different than the existing 4:1. New embankment material strength will increase over time. Complete breaching of the embankment with uncontrolled release of the stored water will probably be more likely to occur in the early stages after construction .

2.6.3.2 Influence of Water Levels

Houston and Duncan used average water levels defined by the Corps of Engineers and a Gumbel distribution to quantify the probability that the maximum slough water level, for any given year, will exceed any specified level. We did not attempt to define the variability of water levels similarly, but selected two conservative combinations for the reservoir and slough water levels in our operational risk analyses. The risk of overtopping will be reduced because of the higher crest elevation of the new embankment (+9), compared with the average existing levee crest (+8).

Excluding the risk of overtopping (which was evaluated in the flood risk analysis; see Section 4.0), Duncan and Houston reported that most historic levee failures have occurred at times of high water in the slough. They postulated that the levee was perhaps pre-weakened by erosion or piping, and pushed inward by the water load on the slough-slide. By placing a new compacted embankment and reservoir on the island-side of the levee, the risk of such an occurrence would be reduced.

Our lowest calculated factors of safety for the long-term condition occur for the slough-side slope, under maximum reservoir level (+4) and low slough water elevation (-1). Such combination was not considered in Duncan and Houston's studies. This water level combination increases seepage forces, the risk of erosion or piping on the slough side, and reduces the effective confining stress and the factor of safety for that slope, compared with the existing condition. Hence, at low slough and high reservoir levels, the risk of slough-side slope failure is higher than without the reservoirs.

Overall, and without overtopping of the embankment, the presence of the embankment and reservoir could lead to higher probabilities of failure on the slough-side. The risk of complete breach of the levee should not be significantly affected, because the embankment will be significantly larger than the existing levee.

2.6.3.3 Influence of Slough Geometry

Houston and Duncan used survey data to define the levee cross sections in their study by six control points. For each island, they selected "..the cross-section with the smallest factor of safety..". We performed slope stability analyses for the average slope geometry for each section investigated. There is significant variability in the slopes encountered on the slough-side. Tables 2-2 and 2-4 show both the average and steepest slopes encountered at the two islands.

We developed histograms showing the distributions of average slough-slide slopes, based on bathymetric data, for each section analyzed. An example of such distribution is shown on Figure 2-10 for Section 1 of Bacon Island. At that section, the flattest-to-steepest slopes range from about 3.7:1 to 1.7:1(H to V). This is consistent with the slope used by Houston and Duncan in their 1978 analyses, 2.3:1, but indicates that steeper slopes exist. For Webb Tract, the steepest slope we used is about 1.4:1, and the steeper than the average slope used by Houston and Duncan, which is about 2.4:1.

We performed sensitivity analyses by making steeper the slough-slide slope in four of our analysis cross-sections. We found that computed factors of safety decrease almost linearly with increasing slope angles, as measured from the horizontal. A 15-degree increase in slope angle lowers the computed factor of safety by about 30 percent. The maximum difference in the steepest slope angle we encountered (1.4:1) and the average slopes used by Houston and Duncan for Bacon Island (2.3:1) is about 12.5 degrees. Hence, the Houston and Duncan factors of safety would be about 23 percent lower based on the use of the steepest slope now identified. Such reduction would increase the calculated number of failures.

2.6.3.4 Influence of Other Factors

We did not attempt to assess the influence of other factors, such as delayed (progressive) slope failures, pumping or other equipment failures, human errors, and catastrophic events other than flood and earthquakes. While these may have little impact on the overall risk, their consideration would increase the probabilities of failure, as computed by Houston and Duncan.

Overall, we estimate that the probability of experiencing slope failures on the slough-side for the completed reservoirs, under normal operating condition, could be up to 50 percent higher than computed in 1978 for the levees. The corresponding probabilities of slope failure would be 0.03 in 40 years at Bacon Island (0.00075 annual probability) and 0.08 in 40 years at Webb Tract (0.0021 annual probability). However, because of the increased cross-sectional area and crest width of the strengthened levee, the probability of complete breaching of the embankments, accompanied by uncontrolled release of water to adjacent islands or flooding of the reservoirs during their seasonal low stage (September to January), should be less than the above numbers.

2.6.4 Comparison with Clifton Court Forebay

We used a report on the safety evaluation of the Clifton Court Forebay Dam (DWR, 1980) to compare factors of safety computed in slope stability analyses. Factors of safety for long-term and sudden drawdown conditions are presented in the DWR report, and are compared with our results in Table 2-9.

TABLE 2-9 FACTORS OF SAFETY COMPARISON WITH CLIFTON COURT FOREBAY DAM

	Factor of Safety		
Load Case	**Bacon Island [1]**	**Webb Tract [1]**	**Clifton Court Forebay [2]**
Long-Term Steady State Toward Slough	1.2	1.2	1.8
Sudden Drawdown Toward Reservoir	1.0	0.9	2.4

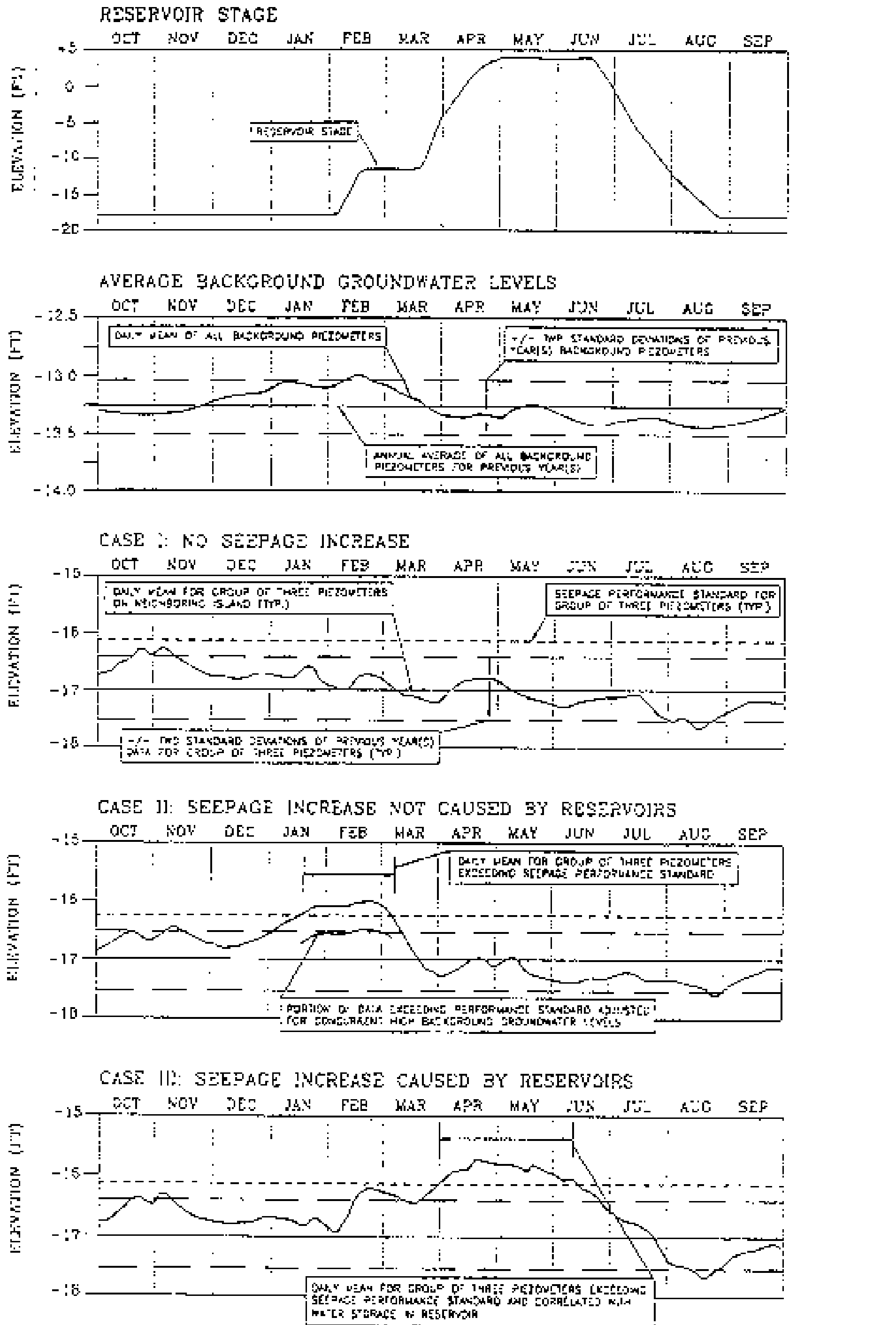
Notes:
1 - Factor of safety are computed using Spencer's procedure
2 - Factor of safety are computed using modified Bishop method

As indicated above, factors of safety for Clifton Court Forebay Dam are higher than those computed in our study. A cursory review of the DWR report indicates that the peat is thinner (15 feet) at Clifton Court Forebay site than at Bacon Island and Webb Tract. Furthermore, drained strength parameters in the Clifton Court analyses and, especially, the effective friction angle (35 degrees) used for the foundation materials, were higher than for our study (26 to 28 degrees). Such observations explain the above differences in computed factors of safety. It should be noted, however, that investigation of the reasons for the differences in strength values (i.e., for peat) was beyond the scope of this work.

2.6.5 Summary

A review of previous studies and additional analyses was conducted to assess the operational risk of the constructed reservoirs at Bacon Island and Webb Tract. These calculations and a qualitative assessment of the facilities to be constructed and associated risk indicate that, under long-term normal operation, the probability of slope failures on the slough-side of the embankment will be increased, compared with the existing levee. This potential problem primarily exists where the channel is deep. The embankments with the existing slopes and a full reservoir have the potential to slide into the channels, which could cause unacceptable environmental damage, damage to floating structures, damage to adjacent levees, potential loss of life, and require expensive dredging to clean up. However, the reservoir-side slopes will stabilize over the long-term and embankments will achieve higher compaction over time. If flatter slopes and a wider crest than for the existing levee are provided, and if sliding did occur on the river/slough side, there would still be enough width until repairs could be made to prevent the loss of reservoir.

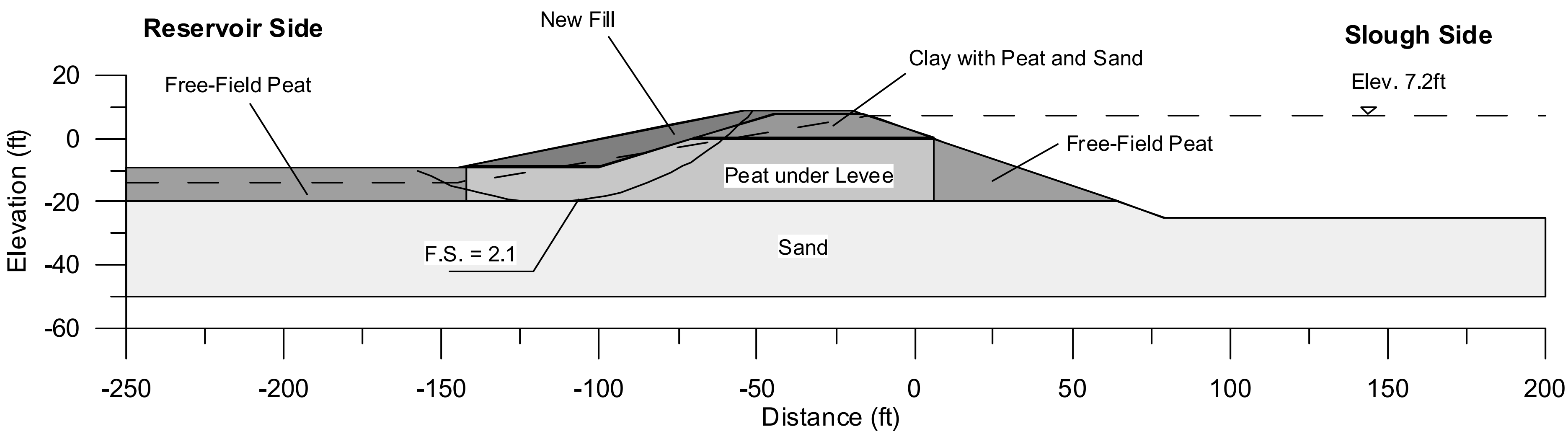
As indicated in the URS (2000) study, the potential for seepage-induced piping and erosion could be high if high water heads are allowed to build behind the levees without seepage control measures. The proposed DW project provides for the construction of the interceptor wells to control adverse seepage conditions. With the proposed interceptor wells system and proper operation and maintenance, this risk will be substantially reduced.



Source: from Hultgren, 1997a

Project No. 41-07099030.00	Delta Wetlands	HYPOTHETICAL PATTERNS OF SEEPAGE RELATIVE TO PERFORMANCE STANDARDS	Figure 2-1
URS Greiner Woodward Clyde			

41-07099030.00-02003/120999/gcs



Type	Unit Weight (pcf)	ϕ'	c' (psf)
1. New Fill	120	30	0
3. Clay with Peat and Sand	110	30	0
4. Peat under Levee	70	28	50
5. Free-Field Peat	70	26	50
7. Sand	125	36	0

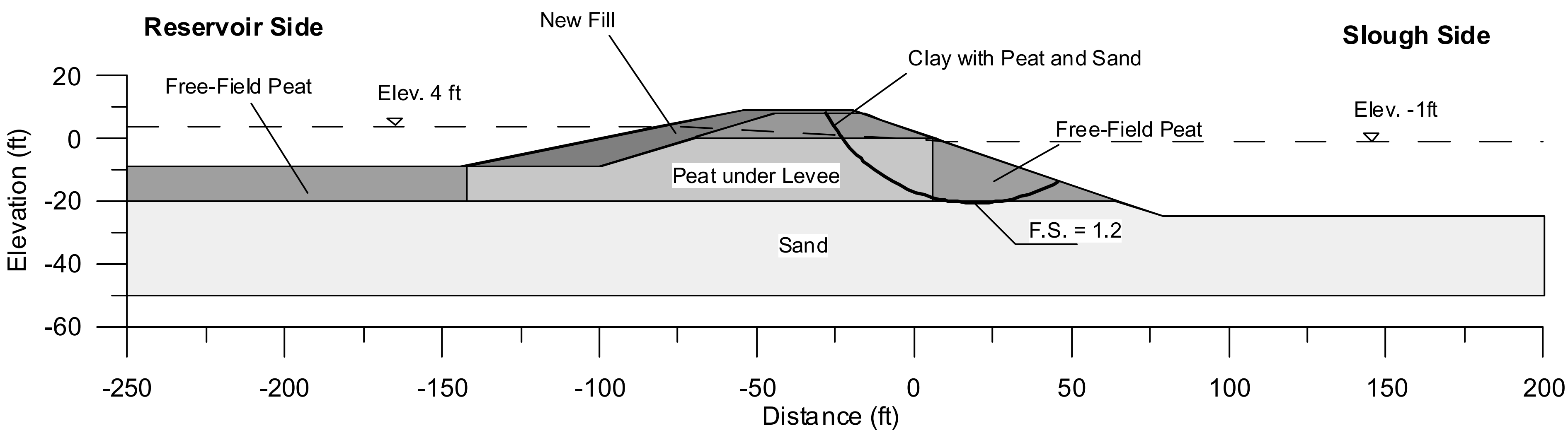
DELTA WETLANDS PROJECT
IN-DELTA STORAGE PRE-FEASIBILITY STUDY

URS

November 2001
Project # 41-F01CS2A6.00

BACON ISLAND-SECTION1
STABILITY ANALYSIS
LONG-TERM CONDITION-TOWARD RESERVOIR

Figure 2-2



	Type	Unit Weight (pcf)	ϕ'	c' (psf)
1.	New Fill	120	30	0
3.	Clay with Peat and Sand	110	30	0
4.	Peat under Levee	70	28	50
5.	Free-Field Peat	70	26	50
7.	Sand	125	36	0

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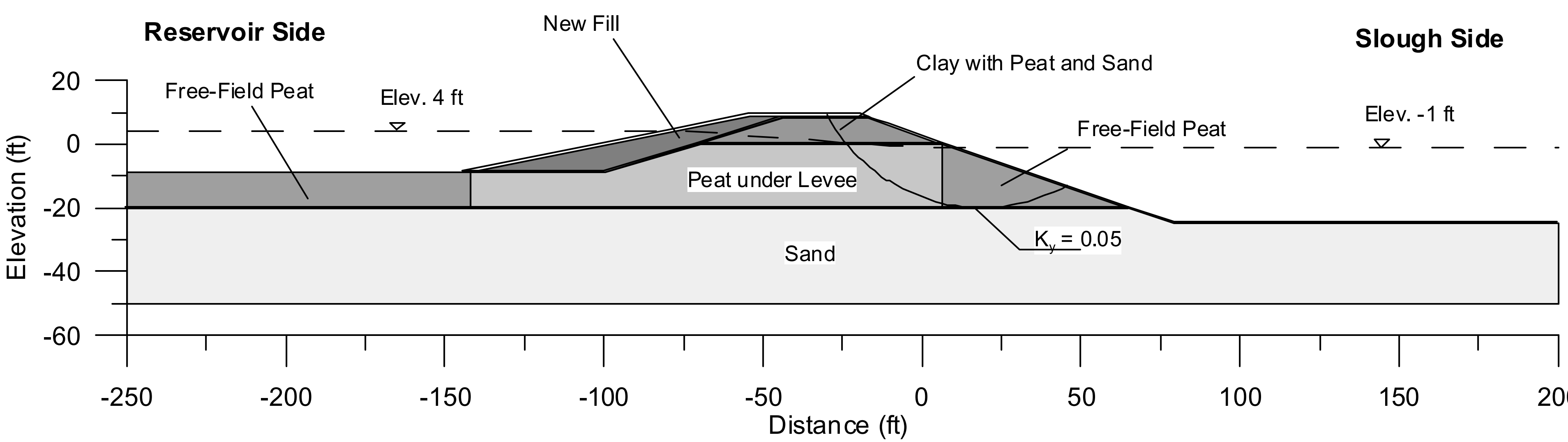
BACON ISLAND-SECTION1
STABILITY ANALYSIS
LONG-TERM CONDITION-TOWARD SLOUGH

Figure 2-3



	Type	Unit Weight (pcf)	φ'	c' (psf)	φ	c (psf)
1.	**New Fill**	120	30	0	30	0
3.	**Clay with Peat and Sand**	110	30	0	30	0
4.	**Peat under Levee**	70	28	50	15	100
5.	**Free-Field Peat**	70	26	50	15	100
7.	**Sand**	125	36	0	36	0

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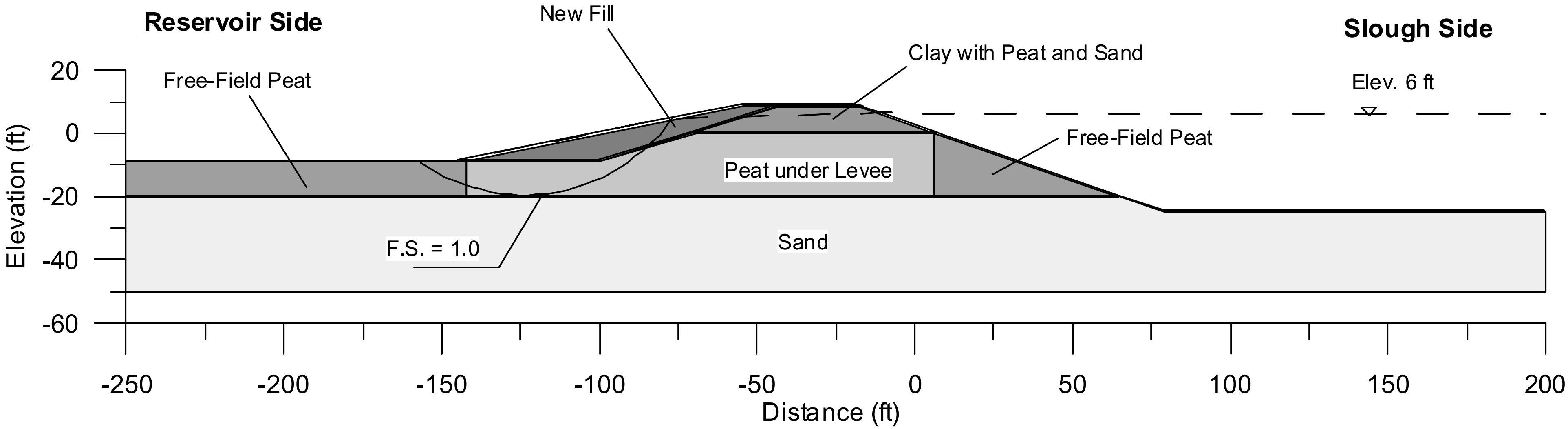
BACON ISLAND-SECTION1
STABILITY ANALYSIS
SEISMIC CONDITION-TOWARD SLOUGH
SCENERIO#1

Figure 2-4



	Type	Unit Weight (pcf)	φ'	c' (psf)	φ	c (psf)
1.	New Fill	120	30	0	30	0
3.	Clay with Peat and Sand	110	30	0	30	0
4.	Peat under Levee	70	28	50	15	100
5.	Free-Field Peat	70	26	50	15	100
7.	Sand	125	36	0	36	0

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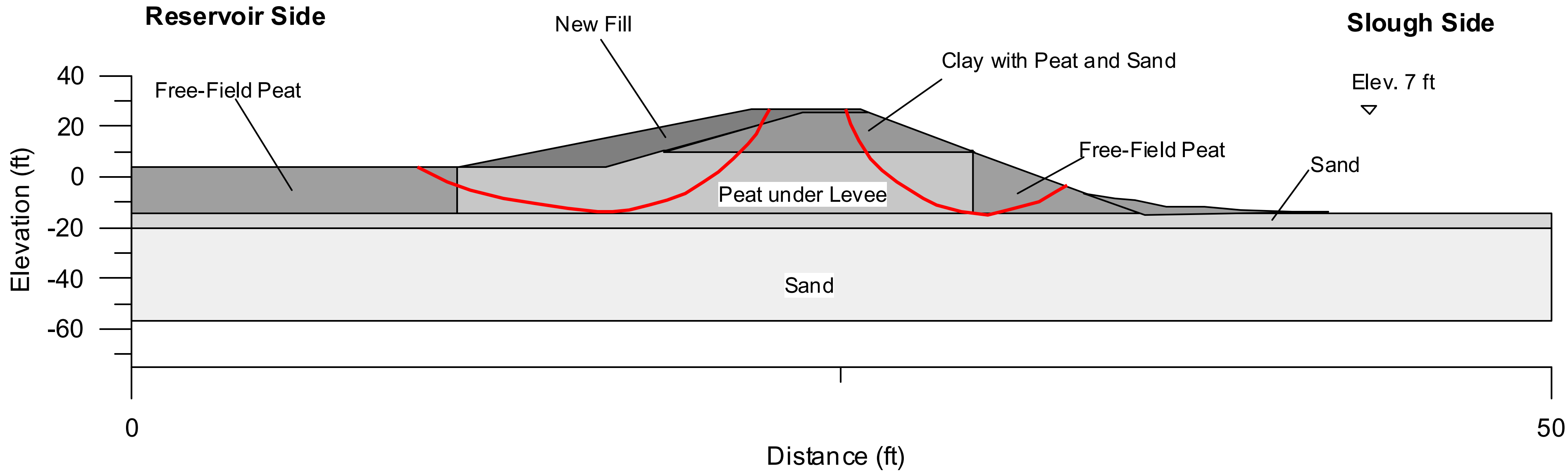
BACON ISLAND-SECTION1
STABILITY ANALYSIS
SUDDEN DRAWDOWN CONDITION
-TOWARD RESERVOIR

Figure 2-5



	Type	Unit Weight (pcf)	φ'	c' (psf)
1.	**New Fill**	120	30	0
3.	**Clay with Peat and Sand**	110	30	0
4.	**Peat under Levee**	70	28	50
5.	**Free-Field Peat**	70	26	50
7.	**Sand**	125	36	0

DELTA WETLANDS PROJECT
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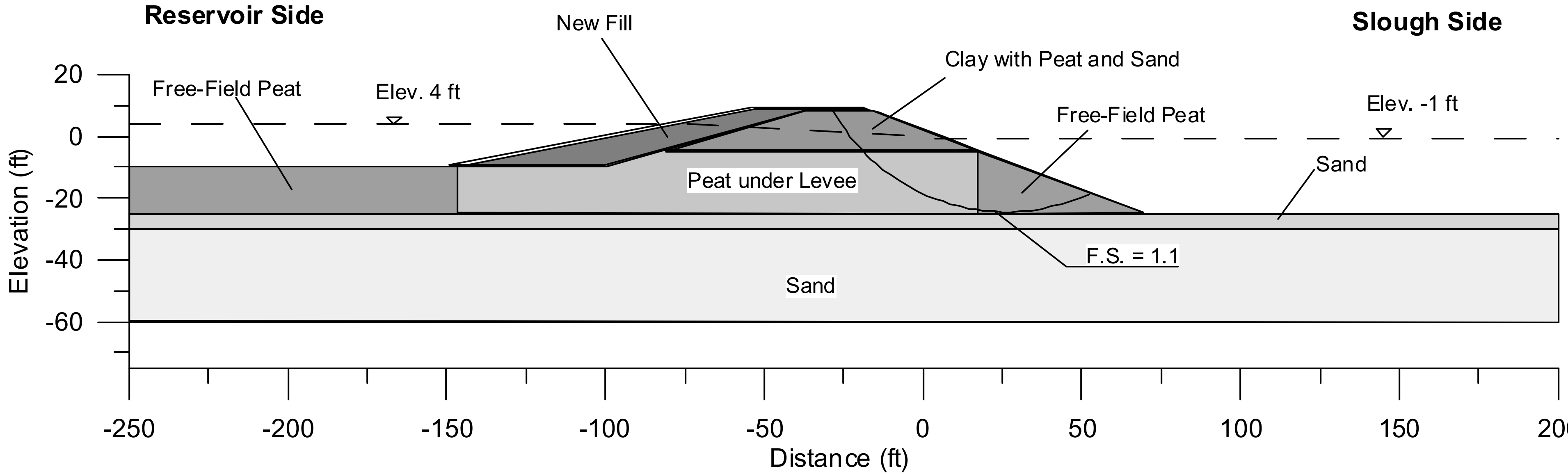
WEBB TRACT ISLAND-SECTION1
STABILITY ANALYSIS
LONG-TERM CONDITION-TOWARD RESERVOIR

Figure 2-6



	Type	Unit Weight (pcf)	ϕ'	c' (psf)
1.	New Fill	120	30	0
3.	Clay with Peat and Sand	110	30	0
4.	Peat under Levee	70	28	50
5.	Free-Field Peat	70	26	50
7.	Sand	125	36	0

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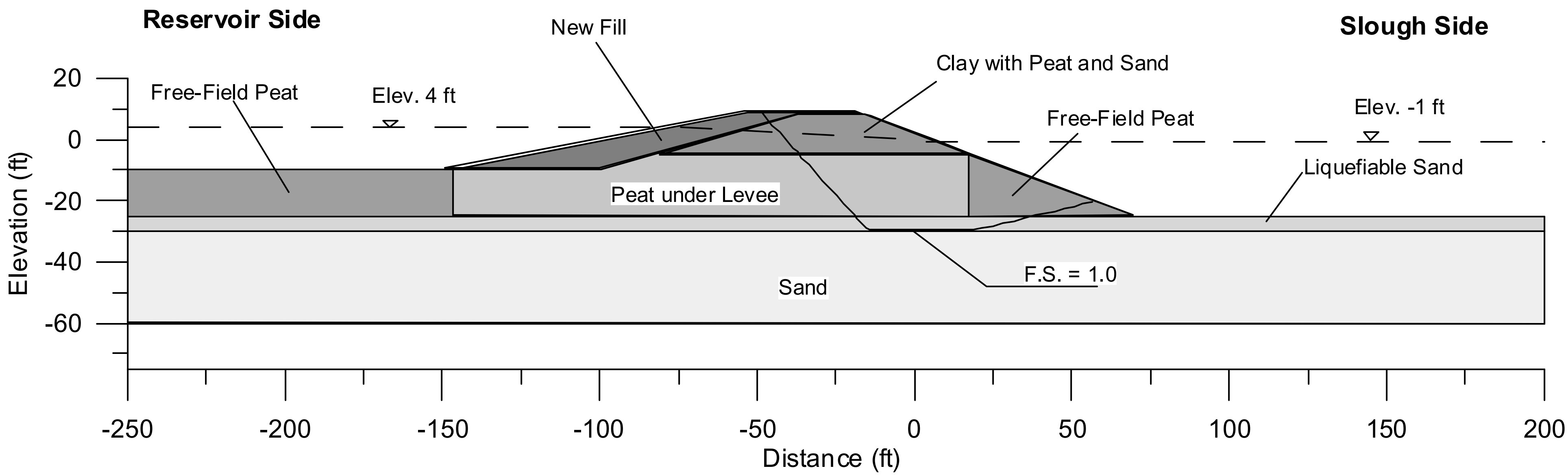
WEBB TRACT ISLAND-SECTION1
STABILITY ANALYSIS
LONG-TERM CONDITION-TOWARD SLOUGH

Figure 2-7



Type	Unit Weight (pcf)	φ'	c' (psf)	φ	c (psf)	S_u (psf)
1. New Fill	120	30	0	30	0	-
3. Clay with Peat and Sand	110	30	0	30	0	-
4. Peat under Levee	70	28	50	15	100	-
5. Free-Field Peat	70	26	50	15	100	-
7. Liquefiable Sand	125	-	-	-	-	200
7. Sand	125	36	0	36	0	-

DELTA WETLANDS PROJECT
IN-DELTA STORAGE PRE-FEASIBILITY STUDY

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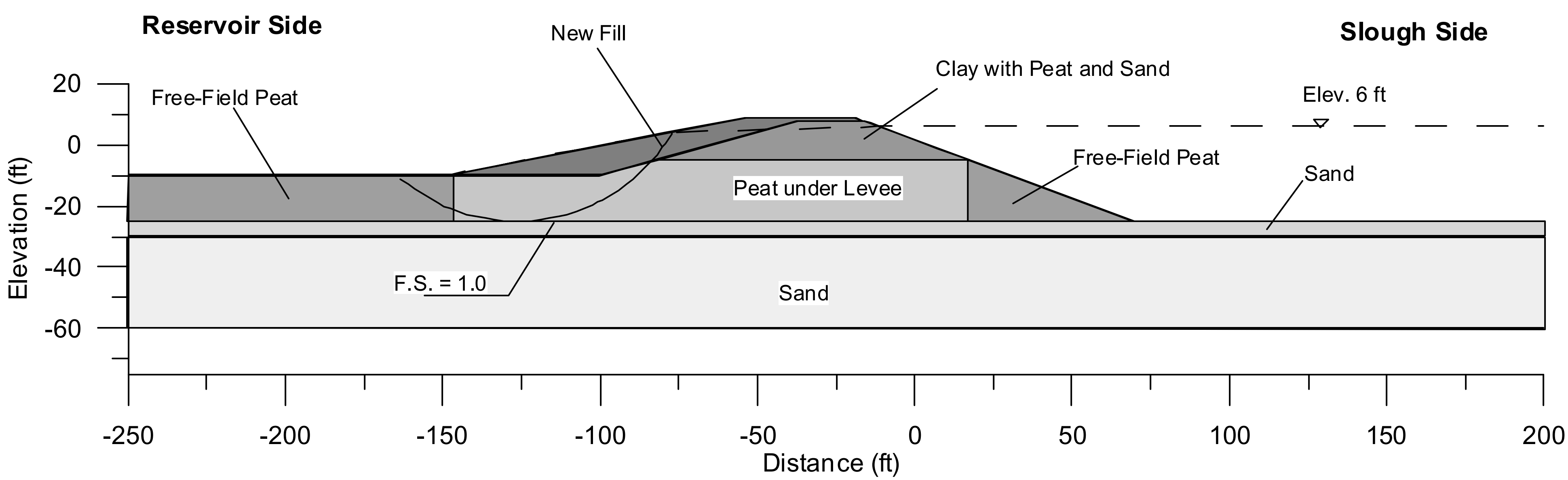
WEBB TRACT ISLAND-SECTION1
STABILITY ANALYSIS
SEISMIC CONDITION-TOWARD SLOUGH
SCENERIO#1

Figure 2-8



	Type	Unit Weight (pcf)	φ'	c' (psf)	φ	c (psf)
1.	New Fill	120	30	0	30	0
3.	Clay with Peat and Sand	110	30	0	30	0
4.	Peat under Levee	70	28	50	15	100
5.	Free-Field Peat	70	26	50	15	100
7.	Sand	125	36	0	36	0

DELTA WETLANDS PROJECT
IN-DELTA STORAGE PRE-FEASIBILITY STUDY

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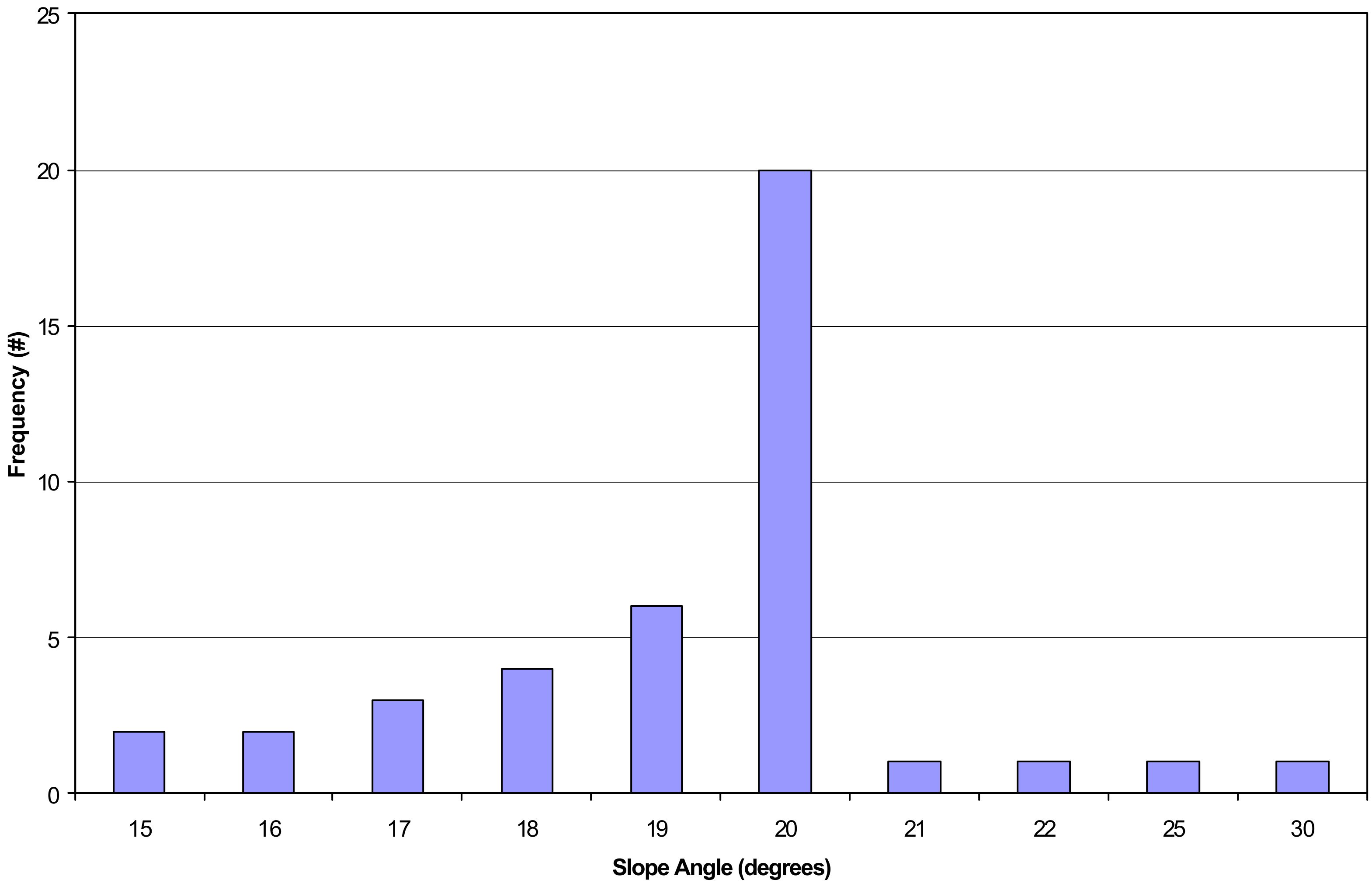
WEBB TRACT ISLAND-SECTION1
STABILITY ANALYSIS
SUDDEN DRAWDOWN CONDITION
-TOWARD RESERVOIR

Figure 2-9

Figure 2-10
Distribution of Slough-Side Slopes for Bacon Island, Section 1



3.1 OBJECTIVE

This section presents the criteria, assumptions, and results of a seismic risk analysis performed for the two proposed reservoir islands for the In-Delta Storage Project. The two proposed islands are the Bacon Island and Webb Tract. The objective of this analysis was to assess the probabilities (risks) of levee failure on these two islands caused by earthquake ground shaking and related effects.

The scope of work calls for a review of previous studies on the Delta Region that are pertinent to our current evaluation. The results of the studies were used to evaluate seismic vulnerability of the levee systems on the two proposed reservoir islands (Bacon Island and Webb Tract).

3.2 REVIEW OF PREVIOUS STUDIES

This study was mainly based on the results of two previous studies on the Delta levees:

- A seismic vulnerability study of the Delta Levees conducted by CALFED (1998), and
- A stability evaluation of the Delta Levees performed by URS (2000).

The CALFED seismic vulnerability study (1998) was performed to assess the seismic risk of the Delta Levees. A team of seismic experts and geotechnical engineers evaluated the seismicity and earthquake ground shaking potential of the Delta Region, developed levee fragility associated with various modes of failure, and assessed seismic vulnerability of the existing levee systems.

As part of the study, a probabilistic seismic hazard analysis (PSHA) was performed for the Delta Region. The results of this analysis are presented in terms of hazard curves for peak ground acceleration (PGA) and ground motion contour maps for a number of return periods at two locations within the study area (Sherman Island and Terminous). Details of the PSHA are given in Appendix A of the CALFED (1998) report.

Levee fragility was evaluated by considering various modes of failure (distress) that lead to uncontrolled flooding of the island. This fragility was expressed as the number of levee breaks per 100 miles of levee length. Both failures associated with liquefaction-induced lateral spreading and earthquake inertia-induced deformations were considered. The study area was divided into four Damage Potential Zones: Zone 1, for high susceptibility, Zone 2, for medium to medium-high susceptibility, and Zones 3 and 4, for low to medium susceptibility. (Zones 3 and 4 have the same ratings for damage potential, but they cover different areas; see Table 4-1 of the CALFED report).

The URS study (2000) considered static and pseudo-static levee stability analyses, groundwater seepage analysis, and levee dynamic response and deformation analyses. The review of the first two analyses is presented in Section 2.0 of this report. Levee dynamic response analysis was performed using a two-dimensional finite element model and an equivalent-linear soil stress-strain model. Newmark double integration method (1965) was used to estimate levee permanent slope deformations.

In the URS study (2000), two levee cross sections for the Bacon Island (at Stations 25+00 and 265+00) and two cross sections for the Webb Tract (at Stations 160+00 and 630+00) were developed for analyses. Earthquake ground motions were represented by two horizontal acceleration time histories: a time history recorded at Station 24402 during the 1987 Whittier Narrows earthquake and a time history recorded at Station 24577 during the 1992 Landers earthquake.

The record from the 1992 Landers earthquake was selected to represent the larger and more distant earthquakes on the San Andreas and Hayward faults. The 1987 Whittier Narrows earthquake was selected to represent the local seismic sources. These time histories were spectrally-matched to the outcropping (near surface) spectrum (URS, 2000).

The results of dynamic response analyses are the computed average acceleration (k_{ave}) time histories for various critical sliding masses. These k_{ave} time histories were used to estimate the permanent deformations of the levees.

Additionally, we have reviewed a report on the liquefaction potential of the Delta levees prepared by the U.S. Army Corps of Engineers (1987). That report identified the occurrences of levee failure during past earthquakes, and rated the Bacon Island and Webb Tract as having medium and high susceptibility to liquefaction, respectively. The report identified seismic-induced levee distress on the northern side of the Webb Tract during recent earthquakes and similar distress on the eastern side of Bacon Island.

3.3 ANALYSIS CRITERIA

The criteria used for the current analysis consist of the following scenarios:

1. Earthquake ground motion scenario.

   Five peak ground accelerations (PGAs) were used for the earthquake scenario. The five selected PGA values have return periods of 43 years, 200 years, 500 years, 1,000 years, and 10,000 years. They correspond to approximately 69%, 22%, 10%, 5%, and 0.5% probabilities of exceedance in 50 years.

   PGA values used for the analyses of Bacon Island and Webb Tract levee systems were taken as equal to those estimated at the "middle" points of the respective islands. PGA contour maps developed as part of the CALFED study (1998) were used for this purpose. Figure 3-1 shows the estimated hazard curves at the "middle" points of the Bacon Island and Webb Tract. The PGA values calculated for the five earthquake ground motion scenarios are tabulated in Tables 3-1 and 3-2, for the Bacon Island and Webb Tract, respectively.

2. Reservoir–slough operating water elevations scenario.

   Three operating water elevation scenarios were selected to represent the annual fluctuation of water elevations in the reservoir and the slough. These selected scenarios are:
   - A full reservoir at elevation +4 feet and slough water at its average low of -1foot. This condition exists during the months of May and June (2 months).
   - An empty reservoir and slough water at its average high of +4 feet. This condition exists from the month of September through the following January (5 months).
   - A half-full reservoir at -8 feet and slough water at its average elevation of +1.8 feet. This condition exists during the months of February through April, when the reservoir is filled, and during the months of July and August, when the reservoir is emptied, for a total of 5 months.

These selected water elevation pairs were estimated from the reservoir stage curve and river stage data showing the hourly fluctuation of slough water elevation. The estimated water elevations in the reservoir and the slough for the selected scenarios are listed in Tables 3-1 and 3-2, for Bacon Island and Webb Tract, respectively, along with their weights. The weights assigned to the various scenarios were estimated proportional to the time periods when the scenarios apply.

3.4 CASES EVALUATED

The Bacon Island and Webb Tract levees were divided into four and three levee sections, respectively, for seismic risk evaluation. These levee sections are the same as those used in operation risk analysis (Section 2.0). The development of selection criteria, section lengths, and typical cross sections of these levee sections are discussed in Section 2.0 of this report.

For each levee section, a seismic risk assessment was conducted using the selected earthquake ground motion and operating water elevation scenarios, as described in Section 3.3. The overall seismic risk for the entire levee system of an island was obtained by summing the risks calculated for the sections.

3.5 ANALYSIS METHODOLOGY/APPROACH

We used a logic tree approach to evaluate the seismic risk of a levee section, as shown in Figure 3-2. For each combination of earthquake ground motion and operating water elevation scenario, the probability of failure of a levee section was estimated as follows:

- Select a yield acceleration coefficient (k_y). The k_y value was estimated for the critical sliding mass identified in the static slope stability analysis (Section 2.0). If liquefaction was expected for the section, k_y was estimated using the residual strength of the liquefied sandy soil; otherwise, k_y was calculated using the undrained shear strengths for clayey soils and peat.
- Tables 3-3 and 3-4 list the yield accelerations calculated for the Bacon Island and Webb Tract, respectively. These yield acceleration values were computed using the soil strength parameters as discussed in Section 2.0.
- Estimate maximum acceleration coefficient (k_{max}) for critical mass. The k_{max} value was taken equal to 0.65*PGA of the outcropping motion. The use of 65% of PGA for k_{max} is based on CALFED recommendation for the Delta Levees (CALFED, 1998).
- Develop average acceleration (k_{ave}) time histories for critical mass. For this pre-feasibility study, we used the average acceleration time histories developed in our previous study (URS, 2000). It was assumed that the results of dynamic response analyses performed during our previous study are representative of the dynamic behavior of the levees.
- As discussed in Section 3.2, these time histories were developed for the Bacon Island and Webb Tract levees using two horizontal earthquake records and two representative cross sections on each island. These average acceleration time histories were then scaled to the k_{max} value estimated above.
- Calculate slope permanent deformations. Using the k_y and k_{ave} time histories developed above, slope permanent deformations were estimated using the Newmark Double Integration Method (Newmark, 1965). We assumed that slough-side slope deformation controls levee

performance. This assumption is consistent with the results of our previous study (URS, 2000) that show the slough-side deformations are larger than those calculated for the reservoir-side slopes.

- Estimate levee failure probability. Consistent with Section 1.2.3 (Task 2.0), Figure B-2 of CALFED report (1998) was used to estimate the levee failure probability, as a function of deformation. Figure B-2 was developed by CALFED Seismic Vulnerability Sub-Team (1998), taking into account the following failure mechanisms:
  - Cracking associated with various deformation levels,
  - Potential exacerbation of seepage problems due to cracking and slumping,
  - Potential overtopping,
  - Potential inboard toe and/or face erosion and piping, and
  - Varying outboard water levels in rivers and sloughs due to both daily tidal fluctuations, and seasonal flow variations.

The overall seismic-induced failure probability of an island levee was then obtained by summing the failure probabilities of sections and then by integrating the results for the various ground motion and operating water elevation scenarios.

3.6 RESULTS OF ANALYSES

Figures 3-3 and 3-4 present the calculated probabilities of levee failure in 50 years, as a function of earthquake peak ground acceleration, for the Bacon Island and Webb Tract, respectively. The results are summarized in Tables 3-5 and 3-6 for the two islands. Appendix 3-A presents further information on the procedures and results of analyses.

The results of seismic vulnerability study indicate that there is about 5.5% chance in 50 years life cycle (0.11% annual probability) that the Bacon Island levee will fail during future earthquakes. The corresponding failure probability for the Webb Tract levee is about 8.5% in 50 years (0.18% annual probability).

It should be noted that for Bacon Island levee, we used yield accelerations that were calculated using the undrained shear strengths. Our review of existing soil borings drilled on the Bacon Island indicates that dense to very dense silty sand exists beneath the peat. Based on an average N-value of 25, we concluded that the post-liquefaction undrained residual shear strength of this silty sand deposit would be higher than the undrained shear strength of the peat (see Section 2.2). Thus, the overall seismic stability of the levees on Bacon Island would be controlled by the low strength of the peat, rather than by the average residual strength of the liquefied sands.

The yield accelerations used for the Webb Tract levees were those calculated using the residual strengths of the liquefied sandy soil deposits. SPT blow-counts recorded during drilling on the Webb Tract are low (an average of 8 blows/ft) and thus, the overall seismic stability of the levees on the Webb Tract would be controlled by the average residual strength of the liquefied sands (see Section 2.0).

In estimating the levee failure probability, we used the empirical relationship developed by CALFED, as shown in Figure B-2 of the CALFED report (1998). It is important to note that this relationship was developed by the CALFED Seismic Vulnerability Sub-Team based on the levee/island operation conditions at the time of that study. We understand that at the time of the CALFED study in 1998, there were no plans to operate the islands as reservoirs. Thus, the failure mechanisms of a levee when the reservoir island is full might be different than those estimated in the CALFED study. Because the probability of levee failure depends largely on this relationship, we recommend that a further evaluation of this relationship be undertaken.

3.7 COMPARISON WITH CLIFTON COURT FOREBAY DAM SEISMIC EVALUATION RESULTS

A safety evaluation was conducted for the nearby Clifton Forebay Dam and Reservoir by the Department of Water Resources (DWR, 1980). In this study, seismic-induced slope deformations were estimated for three maximum credible earthquake magnitudes. They correspond to a magnitude of 8.5 on the San Andreas Fault, a magnitude of 7.3 on the Hayward Fault, and a magnitude of 6.0 on the nearby Marsh Creek-Greenville-Patterson Pass Fault.

Slope deformations were estimated using peak acceleration values predicted at the site from the maximum credible earthquakes and the Makdisi and Seed simplified procedure (1977). The largest deformation was estimated to be less than 1 cm (0.4 inch), an indicator for a satisfactory performance during earthquakes.

The results of this DWR study (1980) indicate lower susceptibility to permanent deformation for the embankment at the Clifton Court Forebay than for that at the Bacon Island and Webb Tract. This is due mainly to a higher yield acceleration (k_y) calculated for the slope at the Clifton Court Forebay. A k_y value of 0.18g was calculated for the Clifton Court Forebay embankment as compared to less than 0.1g for the Bacon Island and Webb Tract. The higher yield acceleration is due to the higher strength parameters used for Clifton Court Forebay as discussed in Section 2.6.4.

TABLE 3-1 EVALUATION CRITERIA FOR BACON ISLAND

EARTHQUAKE GROUND MOTIONS

Ground Motions	Probability Of Exceedance In 50 Year	Return Period	Outcropping Stiff Soil PGA
1	69 %	43 years	0.113 g
2	22 %	200 years	0.189 g
3	10 %	500 years	0.250 g
4	5 %	1,000 years	0.312 g
5	0.5 %	10,000 years	0.640 g

EARTHQUAKE ACCELERATION TIME HISTORIES

Average Acceleration Time History (K_{av})	Earthquake Records	Levee Cross Section	
1	1992 Landers @ St. 24577, 0^0 comp.	Station 25+00	URS (2000) Dynamic Response Analysis Results
2	1992 Landers @ St. 24577, 0^0 comp.	Station 265+00	URS (2000) Dynamic Response Analysis Results
3	1987 Whittier Narrow @ St. 24402, 90^0 comp.	Station 25+00	URS (2000) Dynamic Response Analysis Results
4	1987 Whittier Narrow @ St. 24402, 90^0 comp.	Station 265+00	URS (2000) Dynamic Response Analysis Results

OPERATING WATER ELEVATIONS

Operating Water Conditions	Reservoir Water Elevation	Slough Water Elevation	Time Period (Year)/Weight
1	+ 4 feet	- 1 foot	2 months/0.16
2	Empty	+ 4 feet	5 months/0.42
3	- 8 feet	+ 1.8 feet	5 months/0.42

TABLE 3-2 EVALUATION CRITERIA FOR WEBB TRACT

EARTHQUAKE GROUND MOTIONS

Ground Motions	Probability Of Exceedance In 50 Year	Return Period	Outcropping Stiff Soil PGA
1	69 %	43 years	0.113 g
2	22 %	200 years	0.195 g
3	10 %	500 years	0.263 g
4	5 %	1,000 years	0.360 g
5	0.5 %	10,000 years	0.850 g

EARTHQUAKE ACCELERATION TIME HISTORIES

Average Acceleration Time History (Kav)	Earthquake Records	Levee Cross Section	
1	1992 Landers @ St. 24577, 00 comp.	Station 160+00	URS (2000) Dynamic Response Analysis Results
2	1992 Landers @ St. 24577, 00 comp.	Station 630+00	URS (2000) Dynamic Response Analysis Results
3	1987 Whittier Narrow @ St. 24402, 900 comp.	Station 160+00	URS (2000) Dynamic Response Analysis Results
4	1987 Whittier Narrow @ St. 24402, 900 comp.	Station 630+00	URS (2000) Dynamic Response Analysis Results

OPERATING WATER ELEVATIONS

Operating Water Conditions	Reservoir Water Elevation	Slough Water Elevation	Time Period (Year)/Weight
1	+ 4 feet	- 1 foot	2 months/0.16
2	Empty	+ 4 feet	5 months/0.42
3	- 8 feet	+ 1.8 feet	5 months/0.42

TABLE 3-3 CALCULATED YIELD ACCELERATIONS FOR BACON ISLAND

Section 1

Operating Water Elevation	Water Elevation (ft) Slough	Water Elevation (ft) Reservoir	K_y[1]	Post-Seismic Static F.S.
Scenerio#1	-1	4	0.05	1.21
Scenerio#2	4	Empty	0.095	1.47
Scenerio#3	1.8	-8	0.08	1.36

Section 2

Operating Water Elevation	Water Elevation (ft) Slough	Water Elevation (ft) Reservoir	K_y[1]	Post-Seismic Static F.S.
Scenerio#1	-1	4	0.035	1.16
Scenerio#2	4	Empty	0.068	1.43
Scenerio#3	1.8	-8	0.063	1.34

Section 3

Operating Water Elevation	Water Elevation (ft) Slough	Water Elevation (ft) Reservoir	K_y[1]	Post-Seismic Static F.S.
Scenerio#1	-1	4	0.035	1.14
Scenerio#2	4	Empty	0.08	1.44
Scenerio#3	1.8	-8	0.072	1.33

Section 4

Operating Water Elevation	Water Elevation (ft) Slough	Water Elevation (ft) Reservoir	K_y[1]	Post-Seismic Static F.S.
Scenerio#1	-1	4	0.045	1.19
Scenerio#2	4	Empty	0.075	1.55
Scenerio#3	1.8	-8	0.068	1.42

[1] *K_y values are for slough side slopes*

TABLE 3-4 CALCULATED YIELD ACCELERATIONS FOR WEBB TRACT

Section 1

Operating Water Elevation	Water Elevation (ft) Slough	Water Elevation (ft) Reservoir	K_v[1]	Post-Seismic Static F.S.
Scenerio#1	-1	4	-	0.98
Scenerio#2	4	Empty	0.06	1.3
Scenerio#3	1.8	-8	0.043	1.22

Section 2

Operating Water Elevation	Water Elevation (ft) Slough	Water Elevation (ft) Reservoir	K_v[1]	Post-Seismic Static F.S.
Scenerio#1	-1	4	-	0.96
Scenerio#2	4	Empty	0.05	1.3
Scenerio#3	1.8	-8	0.033	1.18

Section 3

Operating Water Elevation	Water Elevation (ft) Slough	Water Elevation (ft) Reservoir	K_v[1]	Post-Seismic Static F.S.
Scenerio#1	-1	4	0.01	1.06
Scenerio#2	4	Empty	0.078	1.44
Scenerio#3	1.8	-8	0.048	1.28

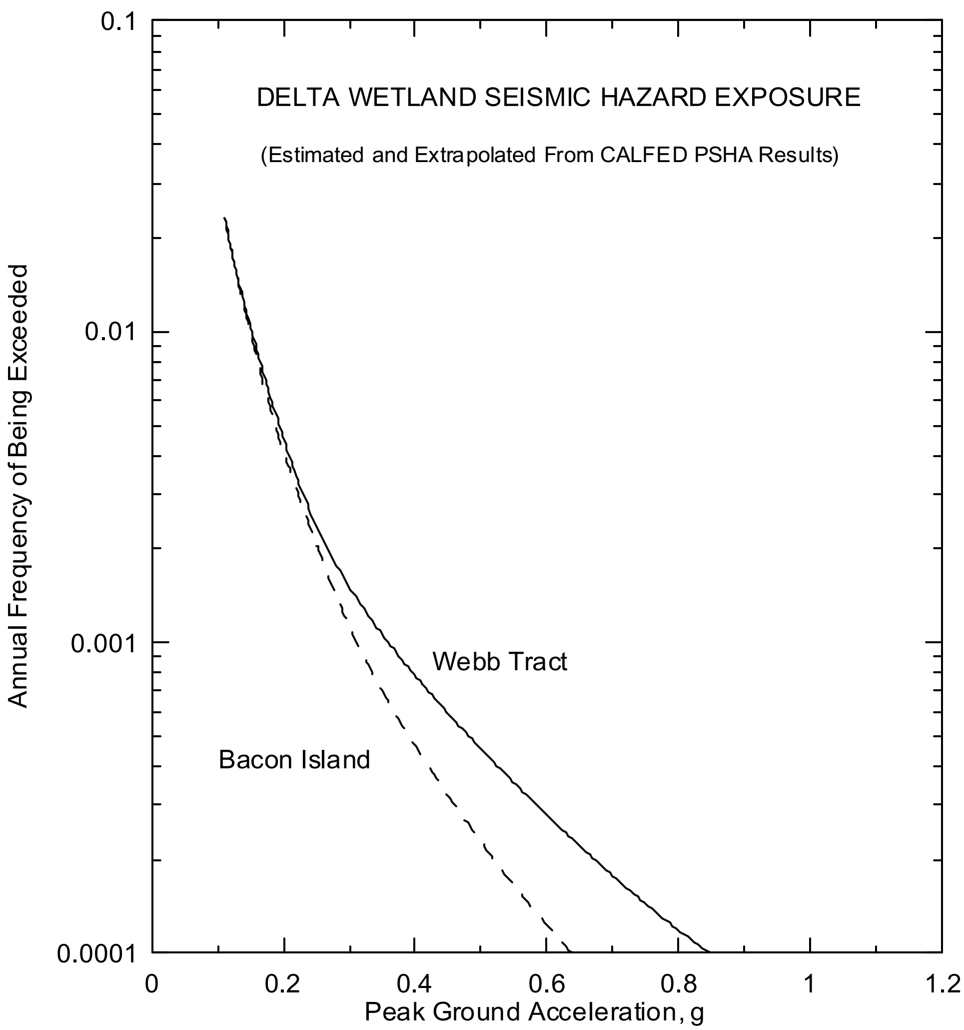
[1] *K_v values are for slough side slopes*

Table 3-5 Summary of Seismic Vulnerability Evaluation for Bacon Island (50-year Life Cycle)

Levee Section	Operating Water Scenario	Weight	Failure Probability, %	Section Failure Probability, %
1	Res +4 ft, Sl -1 ft	0.16	1.56	
	Res Empty, Sl +4	0.42	1.15	
	Res -8 ft, SL +1.8 ft	0.42	1.23	1.25
2	Res +4 ft, Sl -1 ft	0.16	1.93	
	Res Empty, Sl +4	0.42	1.33	
	Res -8 ft, SL +1.8 ft	0.42	1.37	1.44
3	Res +4 ft, Sl -1 ft	0.16	1.93	
	Res Empty, Sl +4	0.42	1.23	
	Res -8 ft, SL +1.8 ft	0.42	1.30	1.37
4	Res +4 ft, Sl -1 ft	0.16	1.65	
	Res Empty, Sl +4	0.42	1.26	
	Res -8 ft, SL +1.8 ft	0.42	1.33	1.35
			Total All Sections	5.41

Table 3-6 Summary of Seismic Vulnerability Evaluation for Webb Tract (50-year Life Cycle)

Levee Section	Operating Water Scenario	Weight	Failure Probability, %	Section Failure Probability, %
1	Res +4 ft, Sl -1 ft	0.16	6.65	
	Res Empty, Sl +4	0.42	1.92	
	Res -8 ft, SL +1.8 ft	0.42	2.23	2.81
2	Res +4 ft, Sl -1 ft	0.16	6.65	
	Res Empty, Sl +4	0.42	2.09	
	Res -8 ft, SL +1.8 ft	0.42	2.56	3.01
3	Res +4 ft, Sl -1 ft	0.16	6.65	
	Res Empty, Sl +4	0.42	1.67	
	Res -8 ft, SL +1.8 ft	0.42	2.12	2.66
			Total All Sections	8.48



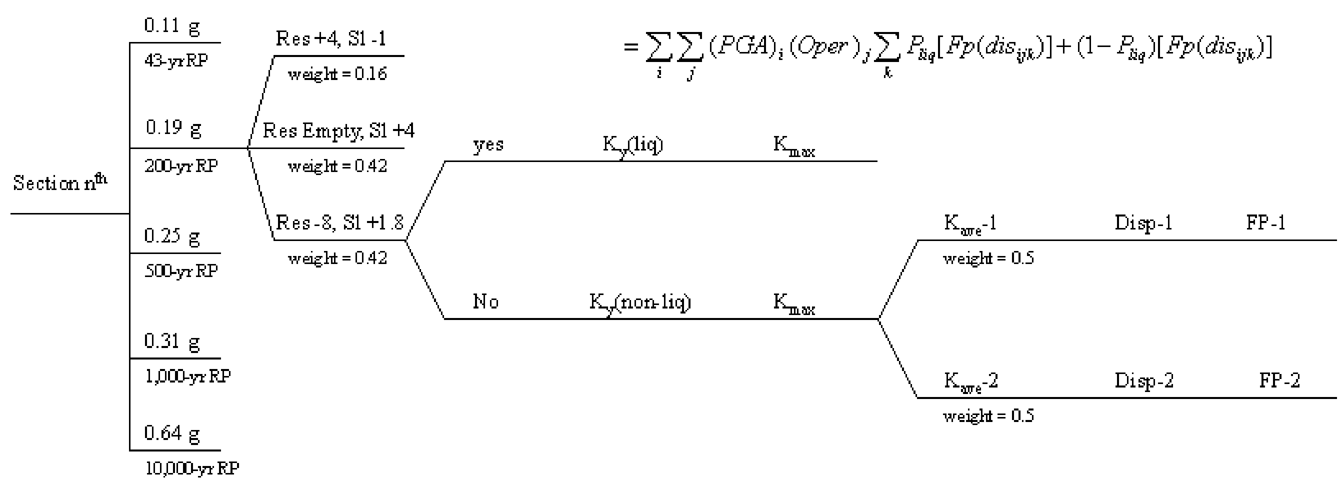
DELTA WETLANDS PROJECT IN-DELTA STORAGE PRE-FEASIBILITY STUDY		Estimated Seismic Hazard Curves for Bacon Island and Webb Tract	**Figure 3-1**
	November 2001 Project # 41-F01CS2A6.00		

SEISMIC VULNERABILITY EVALUATION

Section	PGA	Operating Scenario	Liquefaction Controls Response	Yield Acceleration	Max Ave. Acceleration	Ave. Acceleration Time History	Displace -ment	Failure Probability

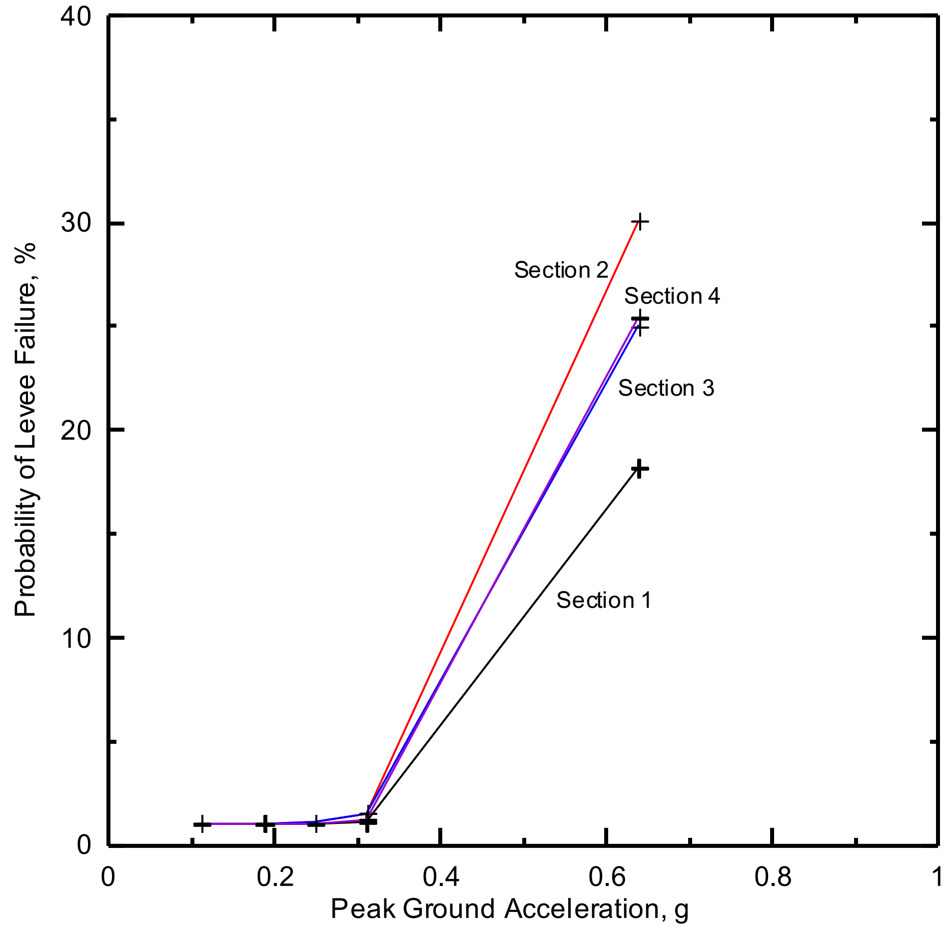
Mean Failure Probability for segment n^{th}

$$= \sum_i \sum_j (PGA)_i (Oper)_j \sum_k P_{liq}[Fp(dis_{ijk})] + (1 - P_{liq})[Fp(dis_{ijk})]$$

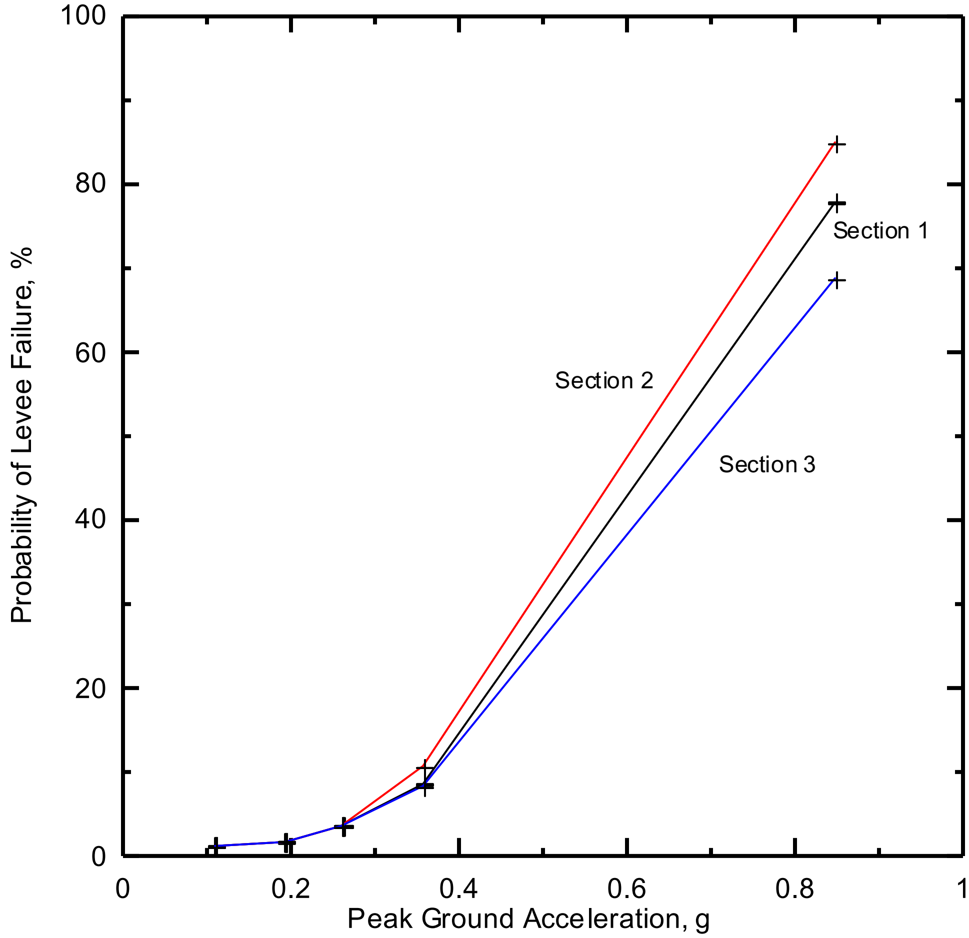


Note: PGA values were estimated using CALFED PSHA results. (Values shown are for Bacon Island)

DELTA WETLANDS PROJECT IN-DELTA STORAGE PRE-FEASIBILITY STUDY URS November 2001 Project # 41-F01CS2A6.00	Logic Tree Used for Seismic Vulnerability Evaluation	**Figure 3-2**



DELTA WETLANDS PROJECT IN-DELTA STORAGE PRE-FEASIBILITY STUDY November 2001 Project # 41-F01CS2A6.00	Levee Failure Probability as Function of Peak Ground Acceleration for Bacon Island	**Figure 3-3**



DELTA WETLANDS PROJECT IN-DELTA STORAGE PRE-FEASIBILITY STUDY	Levee Failure Probability as Function of Peak Ground Acceleration for Webb Tract	Figure 3-4
November 2001 Project # 41-F01CS2A6.00		

4.1 EVALUATION CRITERIA

Probability of flood overtopping failure of the proposed storage reservoir islands at Webb Tract and Bacon Island was evaluated for 100-year flood stages for the Delta Wetlands Project. Tables 4-1 and 4-2 summarize the design criteria related to flood stages and wind-wave runup at the reservoir islands. These flood conditions for the flood risk evaluation were obtained from the report entitled "Levee Rehabilitation Study" by CALFED (1998). Probabilities of flood overtopping failure (R) during the range of design events (T = 50 to 300 years) for the selected project life of 50 years, were also presented for comparison.

In Tables 4-1 and 4-2, the maximum flood elevation is the 100-year flood stage plus the estimated wind-wave runup. In this study, the criteria for evaluating a levee section for flood overtopping risk is the condition of the maximum flood elevation compared to the design levee crest elevation (+9).

The 100-year flood stage and wind-wave runup presented in Tables 4-1 and 4-2 for each of the levee sections were calculated by averaging the values reported in the CALFED study for the stations within each section (see Appendix 4-A). The stations along levee alignments for each reservoir island were grouped into individual sections with similar flood risk conditions. Estimates were grouped by location and by similar flood and wind-wave runup estimates. The extreme values within each section are localized and they are close numerically to the section they are grouped with, thus the average values were presented as a better representation of the flood conditions for each section. The greatest variation between stations is found in the wind-wave runup estimates. These estimates are sensitive to levee slope and fetch length, which vary from station to station. Sections were grouped to incorporate stations with relatively similar adjacent water bodies. Figures 4-1 and 4-2 show the levee section locations.

Table 4-1. Analysis Criteria for Flood Risk Analysis at Webb Tract

Levee Section(1)	Levee Station(1)	100-Year Flood Stage (feet)	Wind-Wave Runup (feet)	Maximum Flood Elevation (feet)
Section 1	0+00 to 70+00	6.8	2.2	8.9
Section 2	70+00 to 220+00	7.0	6.9	13.9
Section 3	220+00 to 290+00	7.1	2.5	9.6
Section 4	290+00 to 350+00	7.0	0.0	7.0
Section 5	350+00 to 590+00	6.8	3.6	10.4
Section 6	590+00 to 680+00	6.7	1.7	8.4

(1). See Figure 4-1 for Levee Section and Station locations.

Table 4-2. Analysis Criteria for Flood Risk Analysis at Bacon Island

Levee Section(1)	Levee Station(1)	100-Year Flood Stage (feet)	Wind-Wave Runup (feet)	Maximum Flood Elevation (feet
Section 1	0+00 to 60+00 700+00 to 750+00	7.3	1.9	9.2
Section 2	60+00 to 112+00	7.3	2.0	9.3
Section 3	112+00 to 130+00	7.3	1.5	8.7
Section 4	130+00 to 135+00	7.2	2.4	9.6
Section 5	135+00 to 159+00	7.2	1.2	8.4
Section 6	159+00 to 170+00	7.2	2.4	9.6
Section 7	170+00 to 250+00	7.2	1.6	8.8
Section 8	250+00 to 280+00	7.1	2.2	9.3
Section 9	280+00 to 320+00	7.1	1.6	8.7
Section 10	320+00 to 350+00	7.1	2.1	9.2
Section 11	350+00 to 570+00	7.2	1.7	8.9
Section 12	570+00 to 580+00	7.3	2.2	9.5
Section 13	580+00 to 610+00	7.3	1.5	8.8
Section 14	610+00 to 630+00	7.3	2.3	9.6
Section 15	630+00 to 700+00	7.4	1.4	8.8

(1). See Figure 4-2 for Levee Section and Station locations.

4.2 METHODOLOGY

Flood overtopping risks at Webb Tract and Bacon Island Reservoirs were evaluated based on the flood stages and wind-wave characteristics estimated for the Sacramento-San Joaquin Delta region. The probability of levee failure due to flood events considered only the potential failure due to overtopping. Other modes of failure, such as seepage-induced failures or slope-stability-induced failures are discussed in Section 2.0, for operational risk.

4.2.1 Flood Stage Analysis

The design flood stage data were obtained from the hydrology study conducted by CALFED (1998). The CALFED study estimated the 100-year flood stages at selected stations along the levee alignments of Bacon Island and Webb Tract reservoir islands. These estimated flood stages adjacent to the reservoir islands are presented in Appendix 4-A.

Tables 4-1 and 4-2 present the 100-year flood stages for the levee sections in Webb Tract and Bacon Island Reservoirs, respectively. Figures 4-1 and 4-2 show the levee sections on plan for Webb Tract and Bacon Island, respectively. The design flood stages for each levee section presented in Tables 4-1 and 4-2 were calculated by averaging the flood stage values estimated in the CALFED study for stations within each section as discussed in Section 4.1.

The average design flood stage data estimated by the CALFED (1998) are in agreement with the 100-year flood stage value reported in the study titled "Sacramento-San Joaquin Delta, California, Special Study, Hydrology" by the U.S. Army Corps of Engineers (USACE, 1992; unpublished). The design flood stage data estimated by CALFED and USACE are summarized in Table 4-3 for comparison. The USACE report (1992) included a stage-frequency analysis. The design floods for the 50-year and 300-year return periods are also included in Table 4-3.

Table 4-3. Analysis Flood Stage Data Estimated by CALFED and USACE

Reservoir Island	Design Flood Stage (feet – NGVD 1929)			
	50-year	100-year		300-year
	USACE	USACE	CALFED(1)	USACE
Webb Tract	6.8	7.0	6.9	7.2
Bacon Island	6.9	7.2	7.2	7.5

(1) Average stage. For specific stage elevation around each island see Appendix 4-A, table A4-1

4.2.2 Wind Wave Analysis

The 100-year flood stage estimates represent the static water level during the flood. These flood stages will increase due to wind wave action on adjacent water bodies. The CALFED (1998) study presents estimated wind-wave conditions at selected stations along the levee alignments of Bacon Island and Webb Tract reservoir islands. These estimated wave runup values at stations along the levee alignments of the reservoir islands are presented in Appendix 4-A. Using these data, the average wave runup values were calculated for each of the levee sections described in Section 4.1. The estimated wave runup values for Webb Tract and Bacon Island reservoirs are summarized in Tables 4-1 and 4-2, respectively.

Table 4-1 shows that the levee section from Station 70+00 to Station 220+00 of Webb Tract Island, located next to Frank's Tract, is subjected to a significant wave runup (6.9 feet) due to wave action on Frank's Tract. The wave runup values at the remaining levee sections range from 0 to 3.6 feet for Webb Tract and 1.2 to 2.4 feet for Bacon Island.

4.2.3 Flood Overtopping

Wave action from wind, including wave setup values (very small effect), are added to the flood stage data to calculate the maximum flood elevations at levees around the reservoir islands. Tables 4-1 and 4-2 summarize the maximum flood elevations for each levee section. The levee crest elevation for both of the proposed reservoir islands at Webb Tract and Bacon Island is 9.0 feet. Tables 4-4 and 4-5 summarize the flood overtopping conditions during the 100-year flood event at Webb Tract and Bacon Island reservoirs, respectively. The overtopping flood depth is the maximum flood elevation minus the design levee crest elevation.

The maximum flood elevation values presented in Table 4-4 and 4-5 are the averages of the values given by CALFED at stations within each section. Sections that consistently have maximum flood elevations below the design levee crest elevation are indicated as "no overtopping" in the comments column in the tables, and for the opposite (overtopping) condition, "flooding". As discussed in Section 4.1, individual estimates of design flood stage within each section vary, and within those sections that are generally close to overtopping flood conditions, there are stations that would be overtopped. However, because these stations share similar flood conditions to the section in which they are grouped, the section was not labeled as "flooding". Instead, such stations were labeled "marginal" for those locations that are close to overtopping.

In some cases, individual stations would have small overtopping flood depths (see Appendix 4-A).

Table 4-4. Flood Overtopping Depths for 100-year Event at Webb Tract

Levee Section(1)	Levee Station(1)	Maximum Flood Elevation (feet)	DW Proposed Levee Crest Elevation (feet)	Overtopping Flood Depth (feet)	Comment
Section 1	0+00 to 70+00	8.8	9.0	-0.2	Marginal
Section 2	70+00 to 220+00	13.9	9.0	4.9	Flooding
Section 3	220+00 to 290+00	9.6	9.0	0.6	Flooding
Section 4	290+00 to 350+00	7.0	9.0	-2.0	No overtopping
Section 5	350+00 to 590+00	10.4	9.0	1.4	Flooding
Section 6	590+00 to 680+00	8.4	9.0	-0.6	Marginal

(1). See Figure 4-1 for Levee Section and Station locations.

Table 4-5. Flood Overtopping Depths for 100-year Event at Bacon Island

Levee Section(1)	Levee Station(1)	Maximum Flood Elevation (feet)	DW Proposed Levee Crest Elevation (feet)	Overtopping Flood Depth (feet)	Comment
Section 1	0+00 to 60+00 700+00 to 750+00	9.2	9.0	0.2	Flooding
Section 2	60+00 to 112+00	9.3	9.0	0.3	Flooding
Section 3	112+00 to 130+00	8.7	9.0	-0.3	No overtopping
Section 4	130+00 to 135+00	9.6	9.0	0.6	Flooding
Section 5	135+00 to 159+00	8.4	9.0	-0.6	Marginal
Section 6	159+00 to 170+00	9.6	9.0	0.6	Flooding
Section 7	170+00 to 250+00	8.8	9.0	-0.2	Marginal
Section 8	250+00 to 280+00	9.3	9.0	0.3	Flooding
Section 9	280+00 to 320+00	8.7	9.0	-0.3	No overtopping
Section 10	320+00 to 350+00	9.2	9.0	0.2	Flooding
Section 11	350+00 to 570+00	8.9	9.0	-0.1	Marginal
Section 12	570+00 to 580+00	9.5	9.0	0.5	Flooding
Section 13	580+00 to 610+00	8.8	9.0	-0.2	No overtopping
Section 14	610+00 to 630+00	9.6	9.0	0.6	Flooding
Section 15	630+00 to 700+00	8.8	9.0	-0.2	No overtopping

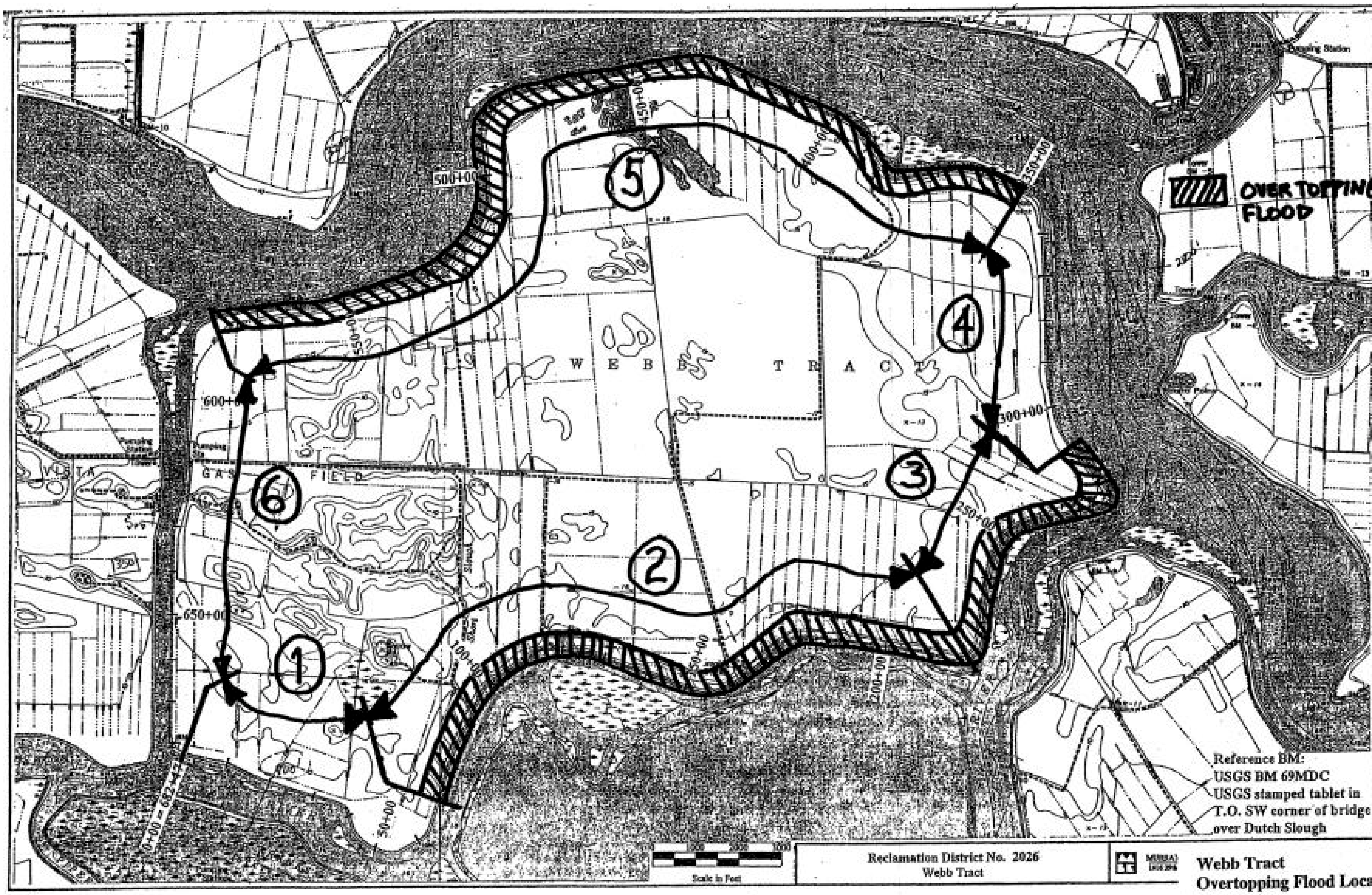
(1). See Figure 4-2 for Levee Section and Station locations.

4.3 RESULTS

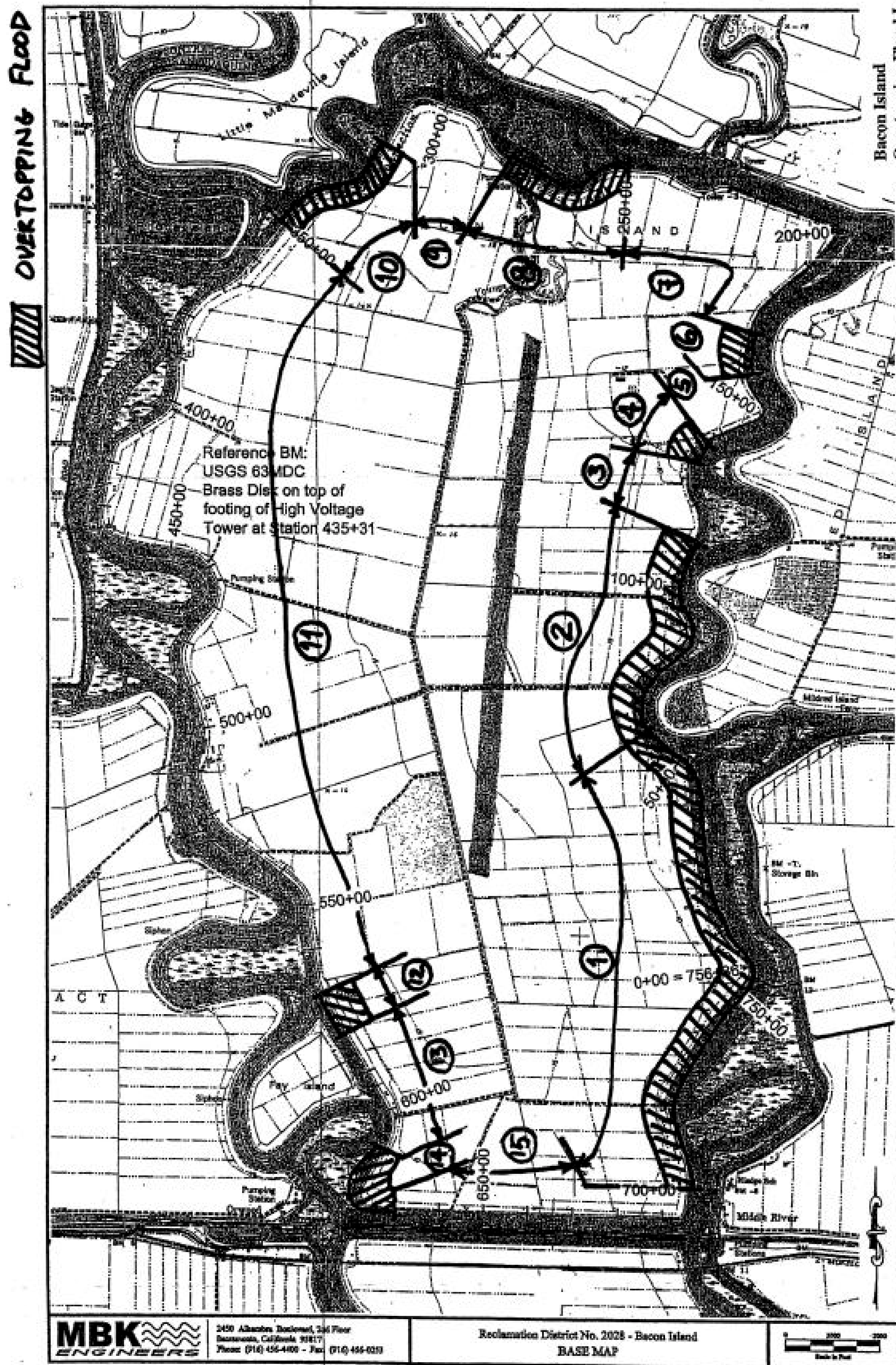
Results summarized in Table 4-4 show that the greatest flood overtopping depth is expected to occur at levee Section 2 of Webb Tract from Station 70+00 to 220+00 (the section adjacent to Frank's Tract). The levee Sections 1 and 5 of Webb Tract are also expected to overtop during the 100-year flood event. Results in Table 4-5 show that eight levee sections at Bacon Island are expected to overtop during the 100-year flood event. The levee sections that are expected to overtop during the 100-year flood event are also shown in Figures 4-1 and 4-2 for Webb Tract and Bacon Island, respectively.

For all sections that overtop, probabilities of overtopping failure during the 100-year flood events were estimated to be 39% for a selected project life of 50 years (about 0.01 annual probability). Figure 4-3 shows the percentage probabilities of flood overtopping failure (R) during range of design events (T = 50 to 300 years) for the selected project life of 50 years. The assumption behind the development of Figure 4-3 is that any time the flood stage and wave run-up is higher than the levee crest, failure due to overtopping is considered to occur.

It should be noted that the wave runup values are strongly dependent on the slope roughness. (Wave runup can be mitigated by rip rap placement.) No information was available from the CALFED (1998) study to determine whether the calculation assumed rip rap on the slough side slopes. We recommend that further investigation in this matter be pursued.

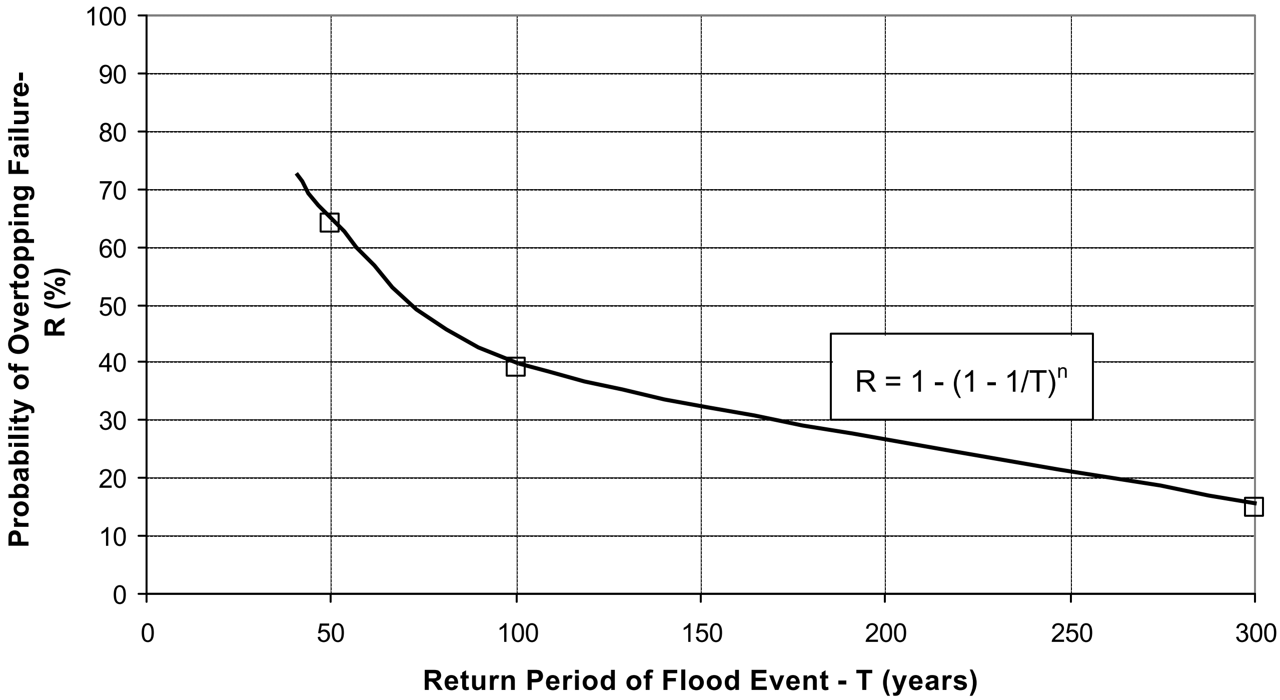


Webb Tract
Overtopping Flood Locations
Figure 4-1



Bacon Island
Overtopping Flood Locations
Figure 4-2

Figure 4-3. Probability of Flood Overtopping Failure for a 50 Year Project Life



5.1 ASSUMPTIONS AND APPROACH

This section was prepared to qualitatively evaluate the consequences of levee failure. For this work, consequences of failure were ranked qualitatively as: high (H), medium (M) or low (L). The information developed to assess the rating of consequences of levee failure were based primarily on the team experience with the Delta, the EIR/EIS document for the Delta Wetlands proposed project, and various relevant published references among which are the Sacramento San Joaquin Delta Atlas (DWR, 1995) and the Calfed Bay Delta program documentation. It is noted that the EIR/EIS document for the Delta Wetlands project addresses only the project impacts and not the impacts resulting from failure.

Table 5-1 was prepared to provide assessment and rating of consequences of levee failure. We have developed the following potentially impacted resources (shown as the column headings in Table 5-1):

- Environmental Impacts
- Impacts to Water Quality and Reliability
- Impacts to Facilities and Infrastructure
- Economic Impacts
- Health and Safety Impacts
- Impact to Land Use

The line headings in Table 5-1 were based on the triggering events evaluated in the previous tasks of this report, and were further broken down into associated failure scenarios. These failure scenarios consisted of defining the slough and reservoir stages during the levee failure. The conditions considered included full reservoir and low slough water, low reservoir and high slough water, and intermediate conditions representing average reservoir and slough water levels. Depending on the triggering event, some of these scenarios may not apply; i.e., during flood event, the low slough water would be high in all cases.

5.2 FINDINGS AND INTERPRETATION OF RESULTS

The findings presented in Table 5-1 and commentary do not include the long-term effects. The long-term effects are those associated with the lasting indirect impacts years after the event and corrective active actions after failure. It should also be noted that during a major earthquake in the region, many other island levees would probably fail and become flooded, and hence the incremental consequences from the DW project failure would be smaller than those indicated in Table 5-1. A similar situation also applies to flood events.

Overall the impact rating to the infrastructure, economy, land use, and health and safety resources are generally medium (M) to low (L) because of the relatively less urbanized affected area with lower asset value compared to urbanized and developed areas. As a result of failure of the Bacon Island embankment due to flood overtopping or seismic failure, a higher potential of damage exists for the Discovery Bay Housing development. High consequences would also be observed in water quality, water supply interruption, and biological resource categories. For water quality, the potential salt-water migration to the discharge pumps during an inward levee breach could affect many water users who would depend on Delta water. For biological resources, some fish species may suffer from entrainment into the reservoir during an inward breach. Fish could be trapped inside the reservoir once the higher slough water starts to recede. The magnitude of impact may vary depending on the fish species and life stage present during migration periods.

Finally, it should be noted that because of the project implementation, it is likely that the reservoir embankments will be stronger. As part of the project implementation, the embankments will be strengthened by flattening the reservoir slopes and widening the crest. However, the proposed crest elevation by Delta Wetlands is +9 feet (MSL). The risk of failure due to flooding overtopping is high. Analysis shows that both island embankments would fail during the 100-year flood event.

Table 5-1
IN-DELTA PRE-FEASIBILITY STUDY
CONSEQUENCES OF FAILURE

TRIGGERING EVENTS	FAILURE SCENARIOS	ENVIRONMENTAL IMPACTS	IMPACTS TO WATER QUALITY & RELIABILITY	IMPACTS TO FACILITIES & INFRASTRUCTURES	ECONOMIC IMPACTS	HEALTH & SAFETY IMPACTS	IMPACT TO LAND USE
SEISMIC EVENT 1	• Reser. full (+4') Slough Low (-1')	M	L-M	M	M	L-M	M
2	• Res. Emp. (-15') Slough High(+6')	H	H	M	L	M	M-H
3	• Avg. Reser. & Slough (+Δ')	L	M	L	L	L	L
4	• Avg. Reser. & Slough (-Δ')	L	M	L	L	L	L
FLOOD EVENT (100-YEAR) 5	• Slough high(+7') Res. emp. (-15')	H	L	M	L	L-M	H
6	• Slough high (+7') Res. half. (-6')	H	L	M	L	L-M	M
7	• Slough high (+7') Res. full. (+4')	L	L	L	L	L	M
OPERATIONAL EVENTS 8	• Res. emp. (-15') Slough high(+7')	H	H	M	L	M	H
9	• Res. Full. (+4') Slough low(-1')	M	L-M	M	M	L-M	M
10	• Intermediate Reser. & Slough (+Δ')	L	M	L	L	L	L
11	• Intermediate Reser. & Slough (-Δ')	L	M	L	L	L	L

TABLE 5-1 COMMENTARY

In this section the impacted elements for each resource category identified in Table 5-1 are presented. This section also attempts to describe the reasons for the rating value selected for each resource category and each triggering event.

Impacts to Environmental Resources

- (H) Entrainment and potential mortality of fish species due to a high water head and inward breach of reservoir dikes.
- (M) Increase in suspended sediments that could affect fish and benthic species in slough channels from an outward breach. Scouring of slough channel and benthic fauna and vegetation along slough channels in vicinity of breach.
- (L) Low water head differential will have a lower potential for fish entrainment and scouring the slough channel.

Impacts to Water Quality And Reliability Resources

- (H) Inward breach would induce migration of salt water into the Delta to the export pumps.
- (M) Outward breach would not worsen salinity but could impact water temperature.
- (L) Flood event with reservoir in operation would likely mask the effects of Bacon Island failure in the short term. In low head differential condition, minimum flow of water either way would have low impact on water quality.

Impacts to Facilities And Infrastructure Resources

Levee failure and outward uncontrolled release may endanger the structural integrity of the neighboring island levees. The pump stations and fish screens that operate reservoir filling and discharge could be damaged if levees fail. Some of the interceptor wells located along the reservoir levees, which control seepage from becoming critical and flooding the neighboring agriculture islands, could be damaged as a result of levee failure. PG&E gas lines may be exposed and damaged in Bacon Island from an inward breach leading to deep scour holes in the reservoir floor. Ferry docks would be damaged on Webb Tract if levees fail

1 M Expect some damage to a adjacent island levees. Failure of Bacon Island Embankment has high potential of damage to the existing and future Discovery Bay housing development. Potential damage to docks, boats. Damage to DW reservoir facilities at breach sites: destruction of interceptor wells on levee, and possibly damage to pumping station, pipes and fish screens. Loss of project beneficial use and water supply.

2 H Deep scour holes can develop as a result of an inward breach, and could cause the following damages:
Possible damage to utilities and gas lines, including dangerous gas line failure.
Possible damage to docks and ships due to high currents in vicinity of breach.
Destruction of interceptor wells, possible damage to pumping station, pipes and fish screens.
Major repair of breaches and scour holes. Loss of project beneficial use.

3, 4 L No major flows into or out of reservoirs.
Damage mostly at point of breach: interceptor wells.
Levee reconstruction.

5 H Consequences similar to 2.

6 M Consequences similar to 2, but to lesser extent.

7 ML Significant surge into reservoir, but consequences only slightly more severe than 3 and 4.

8 H Consequences similar to 2.

9 ML Consequences similar to 7.

10,11 L similar to 3, 4.

Impacts to Economic Resources

Overall Basis - Unlike urban setting, there are no high-value assets (such as buildings with high property value or critical public function) at risk in case of a levee failure and hence none of the failure scenarios is likely to result in high economic impact. The main components of economic impact are the destruction of crops resulting in loss of agricultural income; the cost of repair and restoration of such facilities as pumping stations, interceptor wells, and fish screens; the interruption of water supply resulting in loss of income to the various water utilities and users; and the interruption of recreational activities (such as boating and hunting) resulting in loss of revenues. The power lines are unlikely to be impacted by the rush of water resulting from a levee failure. The gas transmission pipeline may fail during an inward breach of a levee. However, the economic impact in terms of loss of revenue would be low.

Medium (M) level of economic impact – This level of economic impact is assessed for the failure scenario in which the reservoir is full and the slough is low, and the levee failure would cause water to rush outward to adjacent islands. This event could cause loss of some crops and may destroy the equipment that is installed in the body of the levee. Such equipment may include a pumping station, interceptor wells, and fish screens. The recreational activities may be interrupted for several days thus causing a loss of revenues to local businesses. The water supply may be interrupted for several days causing loss of income to various water utilities and users.

Low (L) level of economic impact – This level of economic impact is assessed for the failure scenarios in which the reservoir is empty and the slough is full, or the reservoir and the slough levels are about equal. The main economic impacts would be the destruction of equipment installed in the levee itself and limited interruption of recreational activities. Any loss of crops is likely to be minimal.

Impacts to Health And Safety Resources

1 L-M Significant surge into reservoir.
There may be danger to people in boats and docks near the failure.

There may also be danger to operators if failure is near operations area

2 M There will be a surge and water currents more severe than in 1. The same dangers apply but to a greater extent.
There may be health impacts due to increased salinity (due to saltwater intrusion) to users of exported water

3,4 L No significant impacts

5 M Impacts same as in 2

6 L-M Impacts similar to 2 but to lesser extent

7 L Moderate surge into reservoir but low health and safety impact
There may be danger to operators if failure is near operations area
Similar to 1 but lesser impact.

8 M Similar to but slightly worse than 2

9 L-M Same as 1

10,11 L Same as 3,4 – No significant impact.

Impacts to Land Use Resources

Land use is mostly agriculture with some open space. The most significant impact to neighboring would be when reservoir is high and slough water is low.

1 L possible seepage and high water in adjacent islands due to interruption in pumping the interceptor wells.

2 H land use over large area may be affected by salt water intrusion. Potential flooding of adjacent islands due to high currents affecting the levee integrity.

Possible effects on adjacent island due to interruption in pumping interceptor wells.

3,4 L No significant effect on land uses.

5 H Similar to 2 above.

6 M Similar to 2, but to lesser extent.

7 L Some surge, but no significant land use impacts expected.

8 H Similar to 2.

9 L Similar to 1.

10,11 L Similar to 3,4.

6.1 SUMMARY

The work presented in this study was based on available subsurface and laboratory data that were developed by others. Focused and specialized field investigation and laboratory testing programs to serve the specific needs of this study were not part of the scope of work and may need to be considered in further project development. As described below, the project, as proposed by DW, has some vulnerabilities to events such as seismic and flooding, and does not meet criteria for slope stability on the slough side. The consequences of potential failure and embankment breaching could lead to unacceptable water quality, property damage and life loss. Embankment performance reliability can be improved with appropriate changes such as flatter slopes, wider crest, and possibly higher embankment. These and other solutions leading to overall system improvement are feasible and should be part of subsequent work activities. Below is a summary of main findings and conclusions from the various sections of this report.

6.2 RESERVOIR EMBANKMENT VULNERABILITY

6.2.1 Slope Stability

End-of-Construction

- Based on analyses of the two typical cross-sections for both Webb Tract and Bacon Islands, the factors of safety ranged from 0.6 to 0.9, for embankment slopes at 5H:1V. In such analyses, all new fill was assumed to be placed at once (one construction season) on the existing levees. The above factors of safety show that such one stage construction will not be feasible.
- The probability of embankment failure with release of water from the adjacent slough into the reservoir area would be significant (say greater than 50 percent), if construction proceeds too rapidly or without staging.

Long-term Condition

- For the assumed embankment geometries and subsurface conditions considered, the computed factors of safety for the slough-side slope are about 1.2 at Bacon Island. Factors of safety on the reservoir-side slope are higher, with a lowest computed value of 1.7. At Webb Tract, the lowest computed factors of safety range from 1.1 to 1.2. The lowest reservoir-side slope factor of safety is 1.6.
- The long-term (steady-state) factors of safety meet or exceed all design criteria for sliding towards the island. Factors of safety for sliding toward the river/slough do not meet any of the design criteria. This potential problem primarily exists where the channel is deep. The embankments with the existing slopes and a full reservoir have the potential to slide into the channels, which could cause unacceptable environmental damage, damage to floating structures, damage to adjacent levees, potential loss of life, and require expensive dredging to clean up.

- The probability of slope failures on the slough-side of the embankment will be increased. The study assigned the risk of these failures as marginal to unacceptable.

Sudden Drawdown

- Computed factors of safety for the Webb and Bacon Island reservoirs range from 0.9 to 1.0. These results are based on the conservative assumption that the new fill along the inside perimeter of the embankment would remain fully saturated after the occurrence of the sudden drawdown.
- It is recognized that some sections had inadequate factors of safety for the sudden drawdown condition and revisions to the proposed configuration would be required in these areas.

Comparison with Clifton Court Forebay

- Factors of safety for Clifton Court Forebay Embankments are higher than those computed for the Delta Wetlands Project because of higher strength parameters used in the analysis of the Forebay.

6.2.2 Seepage

- Seepage models indicate water levels under adjacent islands will rise due to the project.
- The potential for seepage-induced piping and erosion could be high if high water heads are allowed to build behind the embankment without seepage control measures. The proposed DW project provides for the construction of the interceptor wells to control adverse seepage conditions.
- A potentially critical seepage condition could result from exceptional events (of very low probability of occurrence). These conditions may prevail due to local power interruptions or major power failures. For example, power loss or grid failures may last from days to weeks, or even months, in some major historic earthquakes. While backup (e.g., diesel operated pumps) is contemplated for the well system, local or distant large earthquakes could cause extended power failures, or even prevent or limit access to the backup pumps for a significant duration of time.

6.2.3 Seismic

- The estimated displacements would result in severe cracking and possible failure from erosion through cracks or an overtopping failure due to slumping and loss of freeboard.
- The results of seismic vulnerability study indicate that there is about 5.5% chance in 50 years (0.11% annual probability) that the Bacon Island levee will fail during future earthquakes. The corresponding failure probability for the Webb Tract levee is about 8.5% in 50 years (0.18% annual probability).

6.2.4 Flooding

- The greatest flood overtopping depth is expected to occur at levee Section 2 of Webb Tract Island from Station 70+00 to 220+00 (the section adjacent to Frank's Tract). The levee Sections 1 and 5 of Webb Tract Island are also expected to overtop during the 100-year flood event.
- Eight levee sections at Bacon Island are expected to overtop during the 100-year flood event.
- For all sections that overtop, probabilities of overtopping failure during the 100-year flood events were estimated to be 39% for a selected project life of 50 years.
- The DW Project proposed crest elevation of +9 (MSL), would meet the crest elevation criteria for the reservoir side only. The crest elevation required to prevent the design flood event on the river/slough side from overtopping the embankments would not be met.

6.2.5 Recommendations

- Factors of safety for sliding toward the river/slough do not meet the slope stability criteria. It is recommended that the crest be wide enough or inside slopes be flattened so that if sliding did occur, there would still be enough embankment width until repairs could be made to prevent loss of reservoir.
- The embankments would be initially constructed with a 30-foot wide crest and have a final crest width of 22 feet after raising due to settlement. The crest width needs to provide for two-way traffic for construction, maintenance, and to facilitate future fill placement to maintain the crest elevation. It is recommended that the 35-foot width be used to facilitate long-term maintenance and repairs.
- To minimize inundation of adjacent islands due to seepage and to avoid potentially critical seepage conditions resulting from exceptional events such as power failures and seismic events, it is recommended that alternative means of seepage control be investigated.
- Attempting to significantly reduce seismic levee fragility will be both difficult and expensive, and that simply making relatively minor geometric modifications will not significantly reduce seismic vulnerability. Improved emergency response plans and measures (including stockpiling critical materials and equipment) should be developed.
- To avoid overtopping due to flooding, crest elevations of embankments for both islands should be raised. The flood data used in the 100-year event were developed by CALFED (1998). It should be noted however, that the wave runup values are strongly dependent on the slope roughness. No information was available from the CALFED (1998) study to determine whether the calculation assumed rip rap on the slough side slopes. Some further investigation in this matter may be warranted.

6.3 RISK ANALYSES

6.3.1 Consequences of Failure of the Delta Wetlands Project

- The operational risk of levee failure will be small compared to seismic and flood risks. The highest potential risk could be expected due to overtopping during a flooding event.
- ***Webb Tract embankment foundation has a higher susceptibility to liquefaction.***
- Overall the impact rating to the infrastructure, economy, land use, and health and safety resources are generally medium to low because of the relatively less urbanized affected area that have lower asset values compared with more urbanized and developed areas.
- As a result of failure of the Bacon Island embankment due to flood overtopping or seismic failure, higher potential of damage exists for the Discovery Bay Housing development.
- High consequences of embankment failure would be observed in water quality and interruption of water supplies. For water quality, the potential salt-water migration to the discharge pumps during an inward embankment breach could affect many water users who would depend on Delta water.
- High consequences of embankment failure would be observed for biological resources, some fish species may suffer from entrainment into the reservoir during an inward breach. Fish could be trapped inside the reservoir once the higher slough water starts to recede. The magnitude of impact may vary depending on the fish species and life stage present during migration periods.

6.3.2 Recommendations

- Solutions should be developed to enhance the reliability of the project to meet the design criteria. As part of this process, focused field investigation and laboratory tests should be developed to address the specific requirements for the desired level of project reliability.

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Appendix 3-A

Calculation Of Levee Failure Probabilities

SEISMIC VULNERABILITY EVALUATION
BACON ISLAND

OPERATING WATER SCENARIO #1 (RESERVOIR @ +4 FT, SLOUGH @ -1 FT)
LIQUEFACTION POTENTIAL : NO
Note: 1 = Displacements less than 2 inches are set equal to 2 inches

Levee Section	Return Period	Ground Motion Scenario: Probability of exceedance in 50 years	Ground Motion Scenario: Outcropping PGA	Probability of PGA in 50 years	Yield Acceleration (ky)	Maximum Average Acceleration (kmax)	Average Acceleration Time History	Displacement[1]	Probability of Failure	Average Prob. Of Failure	Prob. Of Section Failure
	(years)	(%)	(g)	(%)	(g)	(g)		(inches)	(%)	(%)	(%)
1	43	69	0.113	58.00	0.050	0.073	Landers earthquake @ St. 25	2	1.000	1.000	0.58
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	2	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	200	22	0.189	27.40	0.050	0.123	Landers earthquake @ St. 25	2	1.000	1.000	0.27
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	6	1.000		
							Whittier Narrow earthquake @ St. 265	4	1.000		
	500	10	0.250	7.20	0.050	0.163	Landers earthquake @ St. 25	3	1.000	1.060	0.08
							Landers earthquake @ St. 265	5	1.000		
							Whittier Narrow earthquake @ St. 25	13	1.238		
							Whittier Narrow earthquake @ St. 265	9	1.000		
	1,000	5	0.312	6.00	0.050	0.203	Landers earthquake @ St. 25	6	1.000	1.568	0.09
							Landers earthquake @ St. 265	9	1.000		
							Whittier Narrow earthquake @ St. 25	23	3.004		
							Whittier Narrow earthquake @ St. 265	17	1.268		
	10,000	0.5	0.64	1.40	0.050	0.416	Landers earthquake @ St. 25	32	8.696	37.916	0.53
							Landers earthquake @ St. 265	38	14.061		
							Whittier Narrow earthquake @ St. 25	87	70.282		
							Whittier Narrow earthquake @ St. 265	76	58.624		
											1.56 Section Total
2	43	69	0.113	58.00	0.035	0.073	Landers earthquake @ St. 25	2	1.000	1.000	0.58
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	3	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	200	22	0.189	27.40	0.035	0.123	Landers earthquake @ St. 25	3	1.000	1.000	0.27
							Landers earthquake @ St. 265	4	1.000		
							Whittier Narrow earthquake @ St. 25	11	1.000		
							Whittier Narrow earthquake @ St. 265	8	1.000		
	500	10	0.250	7.20	0.035	0.163	Landers earthquake @ St. 25	7	1.000	1.347	0.10
							Landers earthquake @ St. 265	8	1.000		
							Whittier Narrow earthquake @ St. 25	21	2.219		
							Whittier Narrow earthquake @ St. 265	16	1.170		
	1,000	5	0.312	6.00	0.035	0.203	Landers earthquake @ St. 25	11	1.000	4.051	0.24
							Landers earthquake @ St. 265	14	1.153		
							Whittier Narrow earthquake @ St. 25	33	9.516		
							Whittier Narrow earthquake @ St. 265	26	4.533		
	10,000	0.5	0.64	1.40	0.035	0.416	Landers earthquake @ St. 25	43	19.231	52.242	0.73
							Landers earthquake @ St. 265	49	26.032		
							Whittier Narrow earthquake @ St. 25	106	85.386		
							Whittier Narrow earthquake @ St. 265	96	78.318		

											1.93	Section Total
3	43	69	0.113	58.00	0.035	0.073	Landers earthquake @ St. 25	2	1.000	1.000	0.58	
							Landers earthquake @ St. 265	2	1.000			
							Whittier Narrow earthquake @ St. 25	3	1.000			
							Whittier Narrow earthquake @ St. 265	2	1.000			
	200	22	0.189	27.40	0.035	0.123	Landers earthquake @ St. 25	3	1.000	1.000	0.27	
							Landers earthquake @ St. 265	4	1.000			
							Whittier Narrow earthquake @ St. 25	11	1.000			
							Whittier Narrow earthquake @ St. 265	8	1.000			
	500	10	0.250	7.20	0.035	0.163	Landers earthquake @ St. 25	7	1.000	1.347	0.10	
							Landers earthquake @ St. 265	8	1.000			
							Whittier Narrow earthquake @ St. 25	21	2.219			
							Whittier Narrow earthquake @ St. 265	16	1.170			
	1,000	5	0.312	6.00	0.035	0.203	Landers earthquake @ St. 25	11	1.000	4.051	0.24	
							Landers earthquake @ St. 265	14	1.153			
							Whittier Narrow earthquake @ St. 25	33	9.516			
							Whittier Narrow earthquake @ St. 265	26	4.533			
	10,000	0.5	0.64	1.40	0.035	0.416	Landers earthquake @ St. 25	43	19.231	52.242	0.73	
							Landers earthquake @ St. 265	49	26.032			
							Whittier Narrow earthquake @ St. 25	106	85.386			
							Whittier Narrow earthquake @ St. 265	96	78.318			
											1.93	Section Total
4	43	69	0.113	58.00	0.045	0.073	Landers earthquake @ St. 25	2	1.000	1.000	0.58	
							Landers earthquake @ St. 265	2	1.000			
							Whittier Narrow earthquake @ St. 25	2	1.000			
							Whittier Narrow earthquake @ St. 265	2	1.000			
	200	22	0.189	27.40	0.045	0.123	Landers earthquake @ St. 25	2	1.000	1.000	0.27	
							Landers earthquake @ St. 265	3	1.000			
							Whittier Narrow earthquake @ St. 25	7	1.000			
							Whittier Narrow earthquake @ St. 265	5	1.000			
	500	10	0.250	7.20	0.045	0.163	Landers earthquake @ St. 25	4	1.000	1.033	0.07	
							Landers earthquake @ St. 265	6	1.000			
							Whittier Narrow earthquake @ St. 25	15	1.131			
							Whittier Narrow earthquake @ St. 265	11	1.000			
	1,000	5	0.312	6.00	0.045	0.203	Landers earthquake @ St. 25	8	1.000	2.042	0.12	
							Landers earthquake @ St. 265	10	1.000			
							Whittier Narrow earthquake @ St. 25	26	4.533			
							Whittier Narrow earthquake @ St. 265	19	1.636			
	10,000	0.5	0.64	1.40	0.045	0.416	Landers earthquake @ St. 25	35	11.248	42.604	0.60	
							Landers earthquake @ St. 265	42	18.155			
							Whittier Narrow earthquake @ St. 25	93	75.808			
							Whittier Narrow earthquake @ St. 265	82	65.205			
											1.65	Section Total

Total Probability of Levee Failure in 50 years for Bacon Island 7.05 %

SEISMIC VULNERABILITY EVALUATION
BACON ISLAND

OPERATING WATER SCENARIO #2 (RESERVOIR EMPTY, SLOUGH @ +4 FT)
LIQUEFACTION POTENTIAL : NO

Note: 1 = Displacements less than 2 inches are set equal to 2 inches

Levee Section	Ground Motion Scenario: Return Period	Probability of exceedance in 50 years	Outcropping PGA	Probability of PGA in 50 years	Yield Acceleration (ky)	Maximum Average Acceleration (kmax)	Average Acceleration Time History	Displacement[1]	Probability of Failure	Average Prob. Of Failure	Prob. Of Section Failure
	(years)	(%)	(g)	(%)	(g)	(g)		(inches)	(%)	(%)	(%)
1	43	69	0.113	58.00	0.095	0.073	Landers earthquake @ St. 25	2	1.000	1.000	0.58
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	2	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	200	22	0.189	27.40	0.095	0.123	Landers earthquake @ St. 25	2	1.000	1.000	0.27
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	2	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	500	10	0.250	7.20	0.095	0.163	Landers earthquake @ St. 25	2	1.000	1.000	0.07
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	3	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	1,000	5	0.312	6.00	0.095	0.203	Landers earthquake @ St. 25	2	1.000	1.000	0.06
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	7	1.000		
							Whittier Narrow earthquake @ St. 265	4	1.000		
	10,000	0.5	0.64	1.40	0.095	0.416	Landers earthquake @ St. 25	15	1.131	11.374	0.16
							Landers earthquake @ St. 265	20	1.901		
							Whittier Narrow earthquake @ St. 25	51	28.402		
							Whittier Narrow earthquake @ St. 265	38	14.061		
											1.15 Section Total
2	43	69	0.113	58.00	0.068	0.073	Landers earthquake @ St. 25	2	1.000	1.000	0.58
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	2	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	200	22	0.189	27.40	0.068	0.123	Landers earthquake @ St. 25	2	1.000	1.000	0.27
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	3	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	500	10	0.250	7.20	0.068	0.163	Landers earthquake @ St. 25	2	1.000	1.000	0.07
							Landers earthquake @ St. 265	3	1.000		
							Whittier Narrow earthquake @ St. 25	7	1.000		
							Whittier Narrow earthquake @ St. 265	5	1.000		
	1,000	5	0.312	6.00	0.068	0.203	Landers earthquake @ St. 25	3	1.000	1.038	0.06
							Landers earthquake @ St. 265	5	1.000		
							Whittier Narrow earthquake @ St. 25	14	1.153		
							Whittier Narrow earthquake @ St. 265	10	1.000		
	10,000	0.5	0.64	1.40	0.068	0.416	Landers earthquake @ St. 25	24	3.469	24.308	0.34
							Landers earthquake @ St. 265	29	6.445		
							Whittier Narrow earthquake @ St. 25	70	51.620		
							Whittier Narrow earthquake @ St. 265	57	35.698		
											1.33 Section Total

3	43	69	0.113	58.00	0.080	0.073	Landers earthquake @ St. 25	2	1.000	1.000	0.58
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	2	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	200	22	0.189	27.40	0.080	0.123	Landers earthquake @ St. 25	2	1.000	1.000	0.27
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	2	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	500	10	0.250	7.20	0.080	0.163	Landers earthquake @ St. 25	2	1.000	1.000	0.07
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	5	1.000		
							Whittier Narrow earthquake @ St. 265	3	1.000		
	1,000	5	0.312	6.00	0.080	0.203	Landers earthquake @ St. 25	2	1.000	1.000	0.06
							Landers earthquake @ St. 265	4	1.000		
							Whittier Narrow earthquake @ St. 25	10	1.000		
							Whittier Narrow earthquake @ St. 265	7	1.000		
	10,000	0.5	0.64	1.40	0.080	0.416	Landers earthquake @ St. 25	19	1.636	17.651	0.25
							Landers earthquake @ St. 265	24	3.469		
							Whittier Narrow earthquake @ St. 25	61	40.635		
							Whittier Narrow earthquake @ St. 265	48	24.863		
											1.23 Section Total
4	43	69	0.113	58.00	0.075	0.073	Landers earthquake @ St. 25	2	1.000	1.000	0.58
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	2	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	200	22	0.189	27.40	0.075	0.123	Landers earthquake @ St. 25	2	1.000	1.000	0.27
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	2	1.000		
							Whittier Narrow earthquake @ St. 265	2	1.000		
	500	10	0.250	7.20	0.075	0.163	Landers earthquake @ St. 25	2	1.000	1.000	0.07
							Landers earthquake @ St. 265	2	1.000		
							Whittier Narrow earthquake @ St. 25	6	1.000		
							Whittier Narrow earthquake @ St. 265	4	1.000		
	1,000	5	0.312	6.00	0.075	0.203	Landers earthquake @ St. 25	3	1.000	1.000	0.06
							Landers earthquake @ St. 265	4	1.000		
							Whittier Narrow earthquake @ St. 25	12	1.000		
							Whittier Narrow earthquake @ St. 265	8	1.000		
	10,000	0.5	0.64	1.40	0.075	0.416	Landers earthquake @ St. 25	21	2.219	19.872	0.28
							Landers earthquake @ St. 265	26	4.533		
							Whittier Narrow earthquake @ St. 25	64	44.333		
							Whittier Narrow earthquake @ St. 265	51	28.402		
											1.26 Section Total

Total Probability of Levee Failure in 50 years for Bacon Island **4.97** **%**

OPERATING WATER SCENARIO #3 (RESERVOIR @ -8 ft, SLOUGH @ +1.8 FT)
LIQUEFACTION POTENTIAL : NO

Note: 1 = Displacements less than 2 inches are set equal to 2 inches

Average Acceleration Time History	Displacement[1]	Probability of Failure	Average Prob. Of Failure	Prob. Of Section Failure
	(inches)	(%)	(%)	(%)
Landers earthquake @ St. 25	2	1.000	1.000	0.58
Landers earthquake @ St. 265	2	1.000		
Whittier Narrow earthquake @ St. 25	2	1.000		
Whittier Narrow earthquake @ St. 265	2	1.000		
Landers earthquake @ St. 25	2	1.000	1.000	0.27
Landers earthquake @ St. 265	2	1.000		
Whittier Narrow earthquake @ St. 25	2	1.000		
Whittier Narrow earthquake @ St. 265	2	1.000		
Landers earthquake @ St. 25	2	1.000	1.000	0.07
Landers earthquake @ St. 265	2	1.000		
Whittier Narrow earthquake @ St. 25	5	1.000		
Whittier Narrow earthquake @ St. 265	3	1.000		
Landers earthquake @ St. 25	2	1.000	1.000	0.06
Landers earthquake @ St. 265	4	1.000		
Whittier Narrow earthquake @ St. 25	10	1.000		
Whittier Narrow earthquake @ St. 265	7	1.000		
Landers earthquake @ St. 25	19	1.636	17.342	0.24
Landers earthquake @ St. 265	24	3.469		
Whittier Narrow earthquake @ St. 25	60	39.399		
Whittier Narrow earthquake @ St. 265	48	24.863		
				1.23 **Section Total**
Landers earthquake @ St. 25	2	1.000	1.000	0.58
Landers earthquake @ St. 265	2	1.000		

Whittier Narrow earthquake @ St. 25	2	1.000			
Whittier Narrow earthquake @ St. 265	2	1.000			
Landers earthquake @ St. 25	2	1.000	1.000	0.27	
Landers earthquake @ St. 265	2	1.000			
Whittier Narrow earthquake @ St. 25	3	1.000			
Whittier Narrow earthquake @ St. 265	2	1.000			
Landers earthquake @ St. 25	2	1.000	1.000	0.07	
Landers earthquake @ St. 265	3	1.000			
Whittier Narrow earthquake @ St. 25	9	1.000			
Whittier Narrow earthquake @ St. 265	6	1.000			
Landers earthquake @ St. 25	4	1.000	1.043	0.06	
Landers earthquake @ St. 265	6	1.000			
Whittier Narrow earthquake @ St. 25	16	1.170			
Whittier Narrow earthquake @ St. 265	11	1.000			
Landers earthquake @ St. 25	26	4.533	27.352	0.38	
Landers earthquake @ St. 265	31	7.911			
Whittier Narrow earthquake @ St. 25	74	56.330			
Whittier Narrow earthquake @ St. 265	61	40.635			
				1.37	**Section Total**
Landers earthquake @ St. 25	2	1.000	1.000	0.58	
Landers earthquake @ St. 265	2	1.000			
Whittier Narrow earthquake @ St. 25	2	1.000			
Whittier Narrow earthquake @ St. 265	2	1.000			
Landers earthquake @ St. 25	2	1.000	1.000	0.27	
Landers earthquake @ St. 265	2	1.000			
Whittier Narrow earthquake @ St. 25	2	1.000			
Whittier Narrow earthquake @ St. 265	2	1.000			
Landers earthquake @ St. 25	2	1.000	1.000	0.07	
Landers earthquake @ St. 265	2	1.000			
Whittier Narrow earthquake @ St. 25	6	1.000			
Whittier Narrow earthquake @ St. 265	4	1.000			
Landers earthquake @ St. 25	3	1.000	1.060	0.06	
Landers earthquake @ St. 265	5	1.000			
Whittier Narrow earthquake @ St. 25	13	1.238			
Whittier Narrow earthquake @ St. 265	9	1.000			

Landers earthquake @ St. 25	22	2.587	21.936	0.31	
Landers earthquake @ St. 265	27	5.130			
Whittier Narrow earthquake @ St. 25	67	48.002			
Whittier Narrow earthquake @ St. 265	54	32.023			
				1.30	**Section Total**
Landers earthquake @ St. 25	2	1.000	1.000	0.58	
Landers earthquake @ St. 265	2	1.000			
Whittier Narrow earthquake @ St. 25	2	1.000			
Whittier Narrow earthquake @ St. 265	2	1.000			
Landers earthquake @ St. 25	2	1.000	1.000	0.27	
Landers earthquake @ St. 265	2	1.000			
Whittier Narrow earthquake @ St. 25	3	1.000			
Whittier Narrow earthquake @ St. 265	2	1.000			
Landers earthquake @ St. 25	2	1.000	1.000	0.07	
Landers earthquake @ St. 265	3	1.000			
Whittier Narrow earthquake @ St. 25	7	1.000			
Whittier Narrow earthquake @ St. 265	5	1.000			
Landers earthquake @ St. 25	3	1.000	1.038	0.06	
Landers earthquake @ St. 265	5	1.000			
Whittier Narrow earthquake @ St. 25	14	1.153			
Whittier Narrow earthquake @ St. 265	10	1.000			
Landers earthquake @ St. 25	24	3.469	24.308	0.34	
Landers earthquake @ St. 265	29	6.445			
Whittier Narrow earthquake @ St. 25	70	51.620			
Whittier Narrow earthquake @ St. 265	57	35.698			
				1.33	**Section Total**

Total Probability of Levee Failure in 50 years for Bacon Island **5.23** **%**

SUMMARY OF SEISMIC VULNERABILITY EVALUATION
BACON ISLAND

Levee Section	Operating Water Scenario	Weight	Failure Probability, %	Section Failure Probability, %
1	Res +4 ft, Sl -1 ft	0.16	1.56	
	Res Empty, Sl +4	0.42	1.15	
	Res -8 ft, SL +1.8 ft	0.42	1.23	1.25
2	Res +4 ft, Sl -1 ft	0.16	1.93	
	Res Empty, Sl +4	0.42	1.33	
	Res -8 ft, SL +1.8 ft	0.42	1.37	1.44
3	Res +4 ft, Sl -1 ft	0.16	1.93	
	Res Empty, Sl +4	0.42	1.23	
	Res -8 ft, SL +1.8 ft	0.42	1.30	1.37
4	Res +4 ft, Sl -1 ft	0.16	1.65	
	Res Empty, Sl +4	0.42	1.26	
	Res -8 ft, SL +1.8 ft	0.42	1.33	1.35
			Total All Sections	5.41

SEISMIC VULNERABILITY EVALUATION
WEBB TRACT

OPERATING WATER SCENARIO #1 (RESERVOIR @ +4 FT, SLOUGH @ -1 FT)
LIQUEFACTION POTENTIAL : YES
Note: 1 = Displacements less than 2 inches are set equal to 2 inches

	Ground Motion Scenario											
Levee Section	Return Period	Probability of exceedance in 50 years	Outcropping PGA	Probability of PGA in 50 years	Yield Acceleration (ky)	Maximum Average Acceleration (kmax)	Average Acceleration Time History	Displacement[1]	Probability of Failure	Average Prob. Of Failure	Prob. Of Section Failure	
	(years)	(%)	(g)	(%)	(g)	(g)		(inches)	(%)	(%)	(%)	
1	43	69	0.112	58.00	0.010	0.073	Landers earthquake @ St. 160	8	1.000	1.000	0.58	
							Landers earthquake @ St. 630	7	1.000			
							Whittier Narrow earthquake @ St. 160	12	1.000			
							Whittier Narrow earthquake @ St. 630	10	1.000			
	200	22	0.195	26.00	0.010	0.127	Landers earthquake @ St. 160	20	1.901	4.341	1.13	
							Landers earthquake @ St. 630	19	1.636			
							Whittier Narrow earthquake @ St. 160	32	8.696			
							Whittier Narrow earthquake @ St. 630	27	5.130			
	500	10	0.263	9.70	0.010	0.171	Landers earthquake @ St. 160	31	7.911	15.864	1.54	
							Landers earthquake @ St. 630	31	7.911			
							Whittier Narrow earthquake @ St. 160	51	28.402			
							Whittier Narrow earthquake @ St. 630	43	19.231			
	1,000	5	0.36	4.90	0.010	0.234	Landers earthquake @ St. 160	49	26.032	41.621	2.04	
							Landers earthquake @ St. 630	49	26.032			
							Whittier Narrow earthquake @ St. 160	82	65.205			
							Whittier Narrow earthquake @ St. 630	68	49.215			
	10,000	0.5	0.85	1.40	0.010	0.553	Landers earthquake @ St. 160	150	95.148	97.559	1.37	
							Landers earthquake @ St. 630	153	95.089			
							Whittier Narrow earthquake @ St. 160	255	100.000			
							Whittier Narrow earthquake @ St. 630	210	100.000			
											6.65	Section Total
2	43	69	0.112	58.00	0.010	0.073	Landers earthquake @ St. 160	8	1.000	1.000	0.58	
							Landers earthquake @ St. 630	7	1.000			
							Whittier Narrow earthquake @ St. 160	12	1.000			
							Whittier Narrow earthquake @ St. 630	10	1.000			
	200	22	0.195	26.00	0.010	0.127	Landers earthquake @ St. 160	20	1.901	4.341	1.13	
							Landers earthquake @ St. 630	19	1.636			
							Whittier Narrow earthquake @ St. 160	32	8.696			
							Whittier Narrow earthquake @ St. 630	27	5.130			
	500	10	0.263	9.70	0.010	0.171	Landers earthquake @ St. 160	31	7.911	15.864	1.54	
							Landers earthquake @ St. 630	31	7.911			
							Whittier Narrow earthquake @ St. 160	51	28.402			
							Whittier Narrow earthquake @ St. 630	43	19.231			
	1,000	5	0.36	4.90	0.010	0.234	Landers earthquake @ St. 160	49	26.032	41.621	2.04	
							Landers earthquake @ St. 630	49	26.032			
							Whittier Narrow earthquake @ St. 160	82	65.205			
							Whittier Narrow earthquake @ St. 630	68	49.215			
	10,000	0.5	0.85	1.40	0.010	0.553	Landers earthquake @ St. 160	150	95.148	97.559	1.37	
							Landers earthquake @ St. 630	153	95.089			
							Whittier Narrow earthquake @ St. 160	255	100.000			
							Whittier Narrow earthquake @ St. 630	210	100.000			
											6.65	Section Total

3	43	69	0.112	58.00	0.010	0.073	Landers earthquake @ St. 160	8	1.000	1.000	0.58	
							Landers earthquake @ St. 630	7	1.000			
							Whittier Narrow earthquake @ St. 160	12	1.000			
							Whittier Narrow earthquake @ St. 630	10	1.000			
	200	22	0.195	26.00	0.010	0.127	Landers earthquake @ St. 160	20	1.901	4.341	1.13	
							Landers earthquake @ St. 630	19	1.636			
							Whittier Narrow earthquake @ St. 160	32	8.696			
							Whittier Narrow earthquake @ St. 630	27	5.130			
	500	10	0.263	9.70	0.010	0.171	Landers earthquake @ St. 160	31	7.911	15.864	1.54	
							Landers earthquake @ St. 630	31	7.911			
							Whittier Narrow earthquake @ St. 160	51	28.402			
							Whittier Narrow earthquake @ St. 630	43	19.231			
	1,000	5	0.36	4.90	0.010	0.234	Landers earthquake @ St. 160	49	26.032	41.621	2.04	
							Landers earthquake @ St. 630	49	26.032			
							Whittier Narrow earthquake @ St. 160	82	65.205			
							Whittier Narrow earthquake @ St. 630	68	49.215			
	10,000	0.5	0.85	1.40	0.010	0.553	Landers earthquake @ St. 160	150	95.148	97.559	1.37	
							Landers earthquake @ St. 630	153	95.089			
							Whittier Narrow earthquake @ St. 160	255	100.000			
							Whittier Narrow earthquake @ St. 630	210	100.000			
											6.65	**Section Total**

Total Probability of Levee Failure in 50 years for Bacon Island **19.96** **%**

SEISMIC VULNERABILITY EVALUATION
WEBB TRACT

OPERATING WATER SCENARIO #2 (RESERVOIR EMPTY, SLOUGH @ +4 FT)
LIQUEFACTION POTENTIAL : YES

Note: 1 = Displacements less than 2 inches are set equal to 2 inches

Levee Section	Return Period	Ground Motion Scenario Probability of exceedance in 50 years	Outcropping PGA	Probability of PGA in 50 years	Yield Acceleration (ky)	Maximum Average Acceleration (kmax)	Average Acceleration Time History	Displacement[1]	Probability of Failure	Average Prob. Of Failure	Prob. Of Section Failure	
	(years)	(%)	(g)	(%)	(g)	(g)		(inches)	(%)	(%)	(%)	
1	43	69	0.112	58.00	0.060	0.073	Landers earthquake @ St. 160	2	1.000	1.000	0.58	
							Landers earthquake @ St. 630	2	1.000			
							Whittier Narrow earthquake @ St. 160	2	1.000			
							Whittier Narrow earthquake @ St. 630	2	1.000			
	200	22	0.195	26.00	0.060	0.127	Landers earthquake @ St. 160	2	1.000	1.000	0.26	
							Landers earthquake @ St. 630	2	1.000			
							Whittier Narrow earthquake @ St. 160	3	1.000			
							Whittier Narrow earthquake @ St. 630	2	1.000			
									1.000			
	500	10	0.263	9.70	0.060	0.171	Landers earthquake @ St. 160	5		1.000	0.10	
							Landers earthquake @ St. 630	5	1.000			
							Whittier Narrow earthquake @ St. 160	7	1.000			
							Whittier Narrow earthquake @ St. 630	7	1.000			
									1.000			
	1,000	5	0.36	4.90	0.060	0.234	Landers earthquake @ St. 160	12	1.000	1.149	0.06	
							Landers earthquake @ St. 630	12	1.000			
							Whittier Narrow earthquake @ St. 160	18	1.424			
							Whittier Narrow earthquake @ St. 630	16	1.170			
	10,000	0.5	0.85	1.40	0.060	0.553	Landers earthquake @ St. 160	69	50.421	66.526	0.93	
							Landers earthquake @ St. 630	68	49.215			
							Whittier Narrow earthquake @ St. 160	111	88.151			
							Whittier Narrow earthquake @ St. 630	96	78.318			
									1.000		**1.92**	**Section Total**
2	43	69	0.112	58.00	0.050	0.073	Landers earthquake @ St. 160	2	1.000	1.000	0.58	
							Landers earthquake @ St. 630	2	1.000			
							Whittier Narrow earthquake @ St. 160	2	1.000			
							Whittier Narrow earthquake @ St. 630	2	1.000			
	200	22	0.195	26.00	0.050	0.127	Landers earthquake @ St. 160	3	1.000	1.000	0.26	
							Landers earthquake @ St. 630	3	1.000			
							Whittier Narrow earthquake @ St. 160	4	1.000			
							Whittier Narrow earthquake @ St. 630	4	1.000			
									1.000			
	500	10	0.263	9.70	0.050	0.171	Landers earthquake @ St. 160	7	1.000	1.000	0.10	
							Landers earthquake @ St. 630	7	1.000			
							Whittier Narrow earthquake @ St. 160	10	1.000			
							Whittier Narrow earthquake @ St. 630	9	1.000			
	1,000	5	0.36	4.90	0.050	0.234	Landers earthquake @ St. 160	13	1.238	1.898	0.09	
							Landers earthquake @ St. 630	15	1.131			
							Whittier Narrow earthquake @ St. 160	23	3.004			
							Whittier Narrow earthquake @ St. 630	21	2.219			
									1.000			

	10,000	0.5	0.85	1.40	0.050	0.553	Landers earthquake @ St. 160	78	60.871	75.357	1.05	
							Landers earthquake @ St. 630	77	59.754			
							Whittier Narrow earthquake @ St. 160	127	93.696			
							Whittier Narrow earthquake @ St. 630	109	87.106			
											2.09	**Section Total**
3	43	69	0.112	58.00	0.078	0.073	Landers earthquake @ St. 160	2	1.000	1.000	0.58	
							Landers earthquake @ St. 630	2	1.000			
							Whittier Narrow earthquake @ St. 160	2	1.000			
							Whittier Narrow earthquake @ St. 630	2	1.000			
									1.000			
	200	22	0.195	26.00	0.078	0.127	Landers earthquake @ St. 160	2	1.000	1.000	0.26	
							Landers earthquake @ St. 630	2	1.000			
							Whittier Narrow earthquake @ St. 160	2	1.000			
							Whittier Narrow earthquake @ St. 630	2	1.000			
	500	10	0.263	9.70	0.078	0.171	Landers earthquake @ St. 160	3	1.000	1.000	0.10	
							Landers earthquake @ St. 630	3	1.000			
							Whittier Narrow earthquake @ St. 160	4	1.000			
							Whittier Narrow earthquake @ St. 630	3	1.000			
									1.000			
	1,000	5	0.36	4.90	0.078	0.234	Landers earthquake @ St. 160	8	1.000	1.000	0.05	
							Landers earthquake @ St. 630	8	1.000			
							Whittier Narrow earthquake @ St. 160	11	1.000			
							Whittier Narrow earthquake @ St. 630	10	1.000			
	10,000	0.5	0.85	1.40	0.078	0.553	Landers earthquake @ St. 160	56	34.468	49.005	0.69	
							Landers earthquake @ St. 630	52	29.601			
							Whittier Narrow earthquake @ St. 160	89	72.196			
							Whittier Narrow earthquake @ St. 630	77	59.754			
											1.67	**Section Total**

Total Probability of Levee Failure in 50 years for Bacon Island **5.68** **%**

SEISMIC VULNERABILITY EVALUATION
WEBB TRACT

OPERATING WATER SCENARIO #3 (RESERVOIR @ -8 FT, SLOUGH @ +1.8 FT)
LIQUEFACTION POTENTIAL : YES
Note: 1 = Displacements less than 2 inches are set equal to 2 inches

Levee Section	Return Period (years)	Ground Motion Scenario: Probability of exceedance in 50 years (%)	Outcropping PGA (g)	Probability of PGA in 50 years (%)	Yield Acceleration (ky) (g)	Maximum Average Acceleration (kmax) (g)	Average Acceleration Time History	Displacement[1] (inches)	Probability of Failure (%)	Average Prob. Of Failure (%)	Prob. Of Section Failure (%)
1	43	69	0.112	58.00	0.043	0.073	Landers earthquake @ St. 160	2	1.000	1.000	0.58
							Landers earthquake @ St. 630	2	1.000		
							Whittier Narrow earthquake @ St. 160	2	1.000		
							Whittier Narrow earthquake @ St. 630	2	1.000		
	200	22	0.195	26.00	0.043	0.127	Landers earthquake @ St. 160	4	1.000	1.000	0.26
							Landers earthquake @ St. 630	4	1.000		
							Whittier Narrow earthquake @ St. 160	6	1.000		
							Whittier Narrow earthquake @ St. 630	5	1.000		
	500	10	0.263	9.70	0.043	0.171	Landers earthquake @ St. 160	9	1.000	1.000	0.10
							Landers earthquake @ St. 630	8	1.000		
							Whittier Narrow earthquake @ St. 160	11	1.000		
							Whittier Narrow earthquake @ St. 630	12	1.000		
	1,000	5	0.36	4.90	0.043	0.234	Landers earthquake @ St. 160	18	1.424	3.110	0.15
							Landers earthquake @ St. 630	17	1.268		
							Whittier Narrow earthquake @ St. 160	28	5.768		
							Whittier Narrow earthquake @ St. 630	25	3.979		
	10,000	0.5	0.85	1.40	0.043	0.553	Landers earthquake @ St. 160	86	69.299	81.149	1.14
							Landers earthquake @ St. 630	85	68.300		
							Whittier Narrow earthquake @ St. 160	140	95.136		
							Whittier Narrow earthquake @ St. 630	120	91.859		
											2.23 **Section Total**
2	43	69	0.112	58.00	0.033	0.073	Landers earthquake @ St. 160	2	1.000	1.000	0.58
							Landers earthquake @ St. 630	2	1.000		
							Whittier Narrow earthquake @ St. 160	2	1.000		
							Whittier Narrow earthquake @ St. 630	2	1.000		
	200	22	0.195	26.00	0.033	0.127	Landers earthquake @ St. 160	6	1.000	1.000	0.26
							Landers earthquake @ St. 630	6	1.000		
							Whittier Narrow earthquake @ St. 160	9	1.000		
							Whittier Narrow earthquake @ St. 630	8	1.000		
	500	10	0.263	9.70	0.033	0.171	Landers earthquake @ St. 160	10	1.000	1.226	0.12
							Landers earthquake @ St. 630	12	1.000		
							Whittier Narrow earthquake @ St. 160	19	1.636		
							Whittier Narrow earthquake @ St. 630	17	1.268		
	1,000	5	0.36	4.90	0.033	0.234	Landers earthquake @ St. 160	24	3.469	7.204	0.35
							Landers earthquake @ St. 630	22	2.587		
							Whittier Narrow earthquake @ St. 160	38	14.061		
							Whittier Narrow earthquake @ St. 630	32	8.696		

									1.000		
	10,000	0.5	0.85	1.40	0.033	0.553	Landers earthquake @ St. 160	99	80.651	89.088	1.25
							Landers earthquake @ St. 630	99	80.651		
							Whittier Narrow earthquake @ St. 160	163	100.000		
							Whittier Narrow earthquake @ St. 630	138	95.048		
											2.56 Section Total
3	43	69	0.112	58.00	0.048	0.073	Landers earthquake @ St. 160	2	1.000	1.000	0.58
							Landers earthquake @ St. 630	2	1.000		
							Whittier Narrow earthquake @ St. 160	2	1.000		
							Whittier Narrow earthquake @ St. 630	2	1.000		
									1.000		
	200	22	0.195	26.00	0.048	0.127	Landers earthquake @ St. 160	3	1.000	1.000	0.26
							Landers earthquake @ St. 630	3	1.000		
							Whittier Narrow earthquake @ St. 160	5	1.000		
							Whittier Narrow earthquake @ St. 630	4	1.000		
	500	10	0.263	9.70	0.048	0.171	Landers earthquake @ St. 160	8	1.000	1.000	0.10
							Landers earthquake @ St. 630	7	1.000		
							Whittier Narrow earthquake @ St. 160	11	1.000		
							Whittier Narrow earthquake @ St. 630	10	1.000		
	1,000	5	0.36	4.90	0.048	0.234	Landers earthquake @ St. 160	13	1.238	2.234	0.11
							Landers earthquake @ St. 630	15	1.131		
							Whittier Narrow earthquake @ St. 160	25	3.979		
							Whittier Narrow earthquake @ St. 630	22	2.587		
	10,000	0.5	0.85	1.40	0.048	0.553	Landers earthquake @ St. 160	80	63.066	76.893	1.08
							Landers earthquake @ St. 630	79	61.975		
							Whittier Narrow earthquake @ St. 160	131	94.381		
							Whittier Narrow earthquake @ St. 630	111	88.151		
											2.12 Section Total

Total Probability of Levee Failure in 50 years for Bacon Island **6.91** %

SUMMARY OF SEISMIC VULNERABILITY EVALUATION
BACON ISLAND

Levee Section	Operating Water Scenario	Weight	Failure Probability, %	Section Failure Probability, %
1	Res +4 ft, Sl -1 ft	0.16	6.65	
	Res Empty, Sl +4	0.42	1.92	
	Res -8 ft, SL +1.8 ft	0.42	2.23	2.81
2	Res +4 ft, Sl -1 ft	0.16	6.65	
	Res Empty, Sl +4	0.42	2.09	
	Res -8 ft, SL +1.8 ft	0.42	2.56	3.01
3	Res +4 ft, Sl -1 ft	0.16	6.65	
	Res Empty, Sl +4	0.42	1.67	
	Res -8 ft, SL +1.8 ft	0.42	2.12	2.66
			Total All Sections	8.48

Appendix 4-A

Flood Stage And Wind-Wave Runup Data (CALFED 1998)

TABLE 4A-2
Flood Overtopping Depth at Bacon Island during 100-year Event

Levee Section	Station Location	100-Year Flood Stage(1) (ft)	Wind-Wave Runup(1) (ft)	Maximum Flood Elevation (feet)	Overtopping Flood Depth(2) (ft)
Section 1	100	7.3	1.9	9.2	0.2
	1000	7.3	2.7	10	1
	2000	7.3	2.2	9.5	0.5
	3000	7.3	2.3	9.6	0.6
	4000	7.3	2.2	9.5	0.5
	5000	7.3	0	7.3	-1.7
	Average	**7.3**	**1.9**	**9.2**	**0.2**
Section 2	6000	7.3	0	7.3	-1.7
	7000	7.3	1.8	9.1	0.1
	8000	7.3	1.7	9	0
	9000	7.3	2.2	9.5	0.5
	10000	7.3	2.7	10	1
	11000	7.3	1.8	9.1	0.1
	11100	7.3	2	9.3	0.3
	Average	**7.3**	**2.0**	**9.3**	**0.3**
Section 3	11200	7.3	1.3	8.6	-0.4
	11300	7.3	1.3	8.6	-0.4
	11400	7.3	1.3	8.6	-0.4
	11500	7.3	1.3	8.6	-0.4
	11600	7.3	1.3	8.6	-0.4
	11700	7.3	1.3	8.6	-0.4
	11800	7.3	1.3	8.6	-0.4
	11900	7.3	1.3	8.6	-0.4
	12000	7.3	1.4	8.7	-0.3
	12100	7.2	1.6	8.8	-0.2
	12200	7.2	1.7	8.9	-0.1
	12300	7.2	1.7	8.9	-0.1
	12400	7.2	1.7	8.9	-0.1
	12500	7.2	1.7	8.9	-0.1
	12600	7.2	1.7	8.9	-0.1
	12700	7.2	1.6	8.8	-0.2
	12800	7.2	1.5	8.7	-0.3
	12900	7.2	1.8	9	0
	Average	**7.3**	**1.5**	**8.7**	**-0.3**
Section 4	13000	7.2	2	9.2	0.2
	13100	7.2	2.3	9.5	0.5
	13200	7.2	2.5	9.7	0.7
	13300	7.2	2.6	9.8	0.8
	13400	7.2	2.8	10	1
	Average	**7.2**	**2.4**	**9.6**	**0.6**
Section 5	13500	7.2	1.7	8.9	-0.1
	13600	7.2	2.2	9.4	0.4
	13700	7.2	1.6	8.8	-0.2
	13800	7.2	1.8	9	0

TABLE 4A-2
Flood Overtopping Depth at Bacon Island during 100-year Event

Levee Section	Station Location	100-Year Flood Stage[1] (ft)	Wind-Wave Runup[1] (ft)	Maximum Flood Elevation (feet)	Overtopping Flood Depth[2] (ft)
	13900	7.2	1.8	9	0
	14000	7.2	1.7	8.9	-0.1
	14100	7.2	1.8	9	0
	14200	7.2	1.7	8.9	-0.1
	14300	7.2	1.7	8.9	-0.1
	14400	7.2	1.7	8.9	-0.1
	14500	7.2	1.8	9	0
	14600	7.2	1.8	9	0
	14700	7.2	1.8	9	0
	14800	7.2	1.7	8.9	-0.1
	14900	7.2	1.7	8.9	-0.1
	15000	7.2	1.7	8.9	-0.1
	15100	7.2	1.7	8.9	-0.1
	15200	7.2	0	7.2	-1.8
	15300	7.2	0	7.2	-1.8
	15400	7.2	0	7.2	-1.8
	15500	7.2	0	7.2	-1.8
	15600	7.2	0	7.2	-1.8
	15700	7.2	0	7.2	-1.8
	15800	7.2	0	7.2	-1.8
	Average	**7.2**	**1.2**	**8.4**	**-0.6**
Section 6	15900	7.2	2.5	9.7	0.7
	16000	7.2	2.5	9.7	0.7
	16100	7.2	2.6	9.8	0.8
	16200	7.2	2.6	9.8	0.8
	16300	7.2	2.7	9.9	0.9
	16400	7.2	2.7	9.9	0.9
	16500	7.2	2.6	9.8	0.8
	16600	7.2	2.3	9.5	0.5
	16700	7.2	2	9.2	0.2
	16800	7.2	2	9.2	0.2
	16900	7.2	1.90	9.10	0.1
	Average	**7.2**	**2.4**	**9.6**	**0.6**
Section 7	17000	7.2	1.80	9.00	0
	17100	7.2	1.80	9.00	0
	17200	7.2	1.80	9.00	0
	17300	7.2	1.70	8.90	-0.1
	17400	7.2	1.70	8.90	-0.1
	17500	7.2	1.70	8.90	-0.1
	17600	7.2	1.60	8.80	-0.2
	17700	7.2	1.60	8.80	-0.2
	17800	7.2	1.60	8.80	-0.2
	17900	7.2	1.60	8.80	-0.2
	18000	7.2	1.60	8.80	-0.2

TABLE 4A-2
Flood Overtopping Depth at Bacon Island during 100-year Event

Levee Section	Station Location	100-Year Flood Stage(1) (ft)	Wind-Wave Runup(1) (ft)	Maximum Flood Elevation (feet)	Overtopping Flood Depth(2) (ft)
	19000	7.2	1.20	8.40	-0.6
	20000	7.2	1.20	8.40	-0.6
	21000	7.2	1.50	8.70	-0.3
	22000	7.2	1.70	8.90	-0.1
	23000	7.2	1.80	9.00	0
	24000	7.2	1.60	8.80	-0.2
	Average	**7.2**	**1.6**	**8.8**	**-0.2**
Section 8	25000	7.2	2.30	9.50	0.5
	26000	7.1	1.60	8.70	-0.3
	27000	7.1	2.70	9.80	0.8
	Average	**7.1**	**2.2**	**9.3**	**0.3**
Section 9	28000	7.1	1.60	8.70	-0.3
	29000	7.1	1.70	8.80	-0.2
	30000	7.1	1.80	8.90	-0.1
	31000	7.1	1.20	8.30	-0.7
	Average	**7.1**	**1.6**	**8.7**	**-0.3**
Section 10	32000	7.1	2.40	9.50	0.5
	33000	7.1	1.20	8.30	-0.7
	34000	7.1	2.60	9.70	0.7
	Average	**7.1**	**2.1**	**9.2**	**0.2**
Section 11	35000	7.1	1.60	8.70	-0.3
	36000	7.1	1.80	8.90	-0.1
	37000	7.1	1.20	8.30	-0.7
	38000	7.1	2.10	9.20	0.2
	39000	7.1	1.40	8.50	-0.5
	40000	7.2	1.10	8.30	-0.7
	41000	7.2	2.00	9.20	0.2
	42000	7.2	1.90	9.10	0.1
	43000	7.2	1.80	9.00	0
	44000	7.2	1.70	8.90	-0.1
	45000	7.2	1.70	8.90	-0.1
	46000	7.2	1.70	8.90	-0.1
	46100	7.2	1.70	8.90	-0.1
	46200	7.2	1.7	8.9	-0.1
	46300	7.2	1.7	8.9	-0.1
	46400	7.2	1.7	8.9	-0.1
	46500	7.2	1.7	8.9	-0.1
	46600	7.2	1.7	8.9	-0.1
	46700	7.2	1.7	8.9	-0.1
	46800	7.2	1.7	8.9	-0.1
	46900	7.2	1.7	8.9	-0.1
	47000	7.2	1.7	8.9	-0.1
	48000	7.2	1.8	9	0
	49000	7.2	1.9	9.1	0.1

TABLE 4A-2
Flood Overtopping Depth at Bacon Island during 100-year Event

Levee Section	Station Location	100-Year Flood Stage(1) (ft)	Wind-Wave Runup(1) (ft)	Maximum Flood Elevation (feet)	Overtopping Flood Depth(2) (ft)
	50000	7.2	1.8	9	0
	51000	7.2	1.8	9	0
	52000	7.2	1.8	9	0
	53000	7.2	1.8	9	0
	54000	7.2	1.8	9	0
	55000	7.3	1.8	9.1	0.1
	56000	7.3	1.2	8.5	-0.5
	Average	**7.2**	**1.7**	**8.9**	**-0.1**
Section 12	57000	7.3	2.2	9.5	0.5
	Average	**7.3**	**2.2**	**9.5**	**0.5**
Section 13	58000	7.3	1.4	8.7	-0.3
	59000	7.3	1.4	8.7	-0.3
	60000	7.3	1.6	8.9	-0.1
	Average	**7.3**	**1.5**	**8.8**	**-0.2**
Section 14	61000	7.3	2.6	9.9	0.9
	62000	7.3	2	9.3	0.3
	Average	**7.3**	**2.3**	**9.6**	**0.6**
Section 15	63000	7.3	1.4	8.7	-0.3
	64000	7.3	1.4	8.7	-0.3
	65000	7.4	1.4	8.8	-0.2
	66000	7.4	1.4	8.8	-0.2
	67000	7.4	1.4	8.8	-0.2
	68000	7.4	1.4	8.8	-0.2
	69000	7.4	1.4	8.8	-0.2
	Average	**7.4**	**1.4**	**8.8**	**-0.2**
Section 1	70000	7.4	2	9.4	0.4
	71000	7.4	2.4	9.8	0.8
	72000	7.4	1.7	9.1	0.1
	73000	7.4	1.7	9.1	0.1
	74000	7.4	1.8	9.2	0.2
	75000	7.3	1.8	9.1	0.1
	Average	**7.3**	**1.9**	**9.2**	**0.2**

(1). Levee Rehabilitation Study by CALFED (August, 1998)
(2). Levee Crest Elevation is 9.0 feet (DW, 2001)

TABLE 4A-1
Flood Overtopping Depth at Webb Track during 100-year Event

Levee Section	Station Location	100-Year Flood Stage(1) (ft)	Wind-Wave Runup(1) (ft)	Maximum Flood Elevation (feet)	Flood Overtopping Depth(2) (ft)
Section 1	100	6.7	2.1	8.8	-0.2
	1000	6.7	2.1	8.8	-0.2
	2000	6.7	2	8.7	-0.3
	3000	6.8	2.4	9.2	0.2
	4000	6.8	2.5	9.3	0.3
	5000	6.8	2	8.8	-0.2
	6000	6.8	2.1	8.9	-0.1
	Average	**6.8**	**2.2**	**8.9**	**-0.1**
Section 2	7000	6.8	7.7	14.5	5.5
	8000	6.8	7.6	14.4	5.4
	9000	6.9	7.7	14.6	5.6
	9983	6.9	7.9	14.8	5.8
	11000	6.9	7.9	14.8	5.8
	12000	6.9	8.2	15.1	6.1
	13000	6.9	7.7	14.6	5.6
	14000	7	7.6	14.6	5.6
	14900	7	7.4	14.4	5.4
	16000	7	6.5	13.5	4.5
	17000	7	5.6	12.6	3.6
	18000	7	5.7	12.7	3.7
	19000	7	6	13	4
	20000	7.1	5.5	12.6	3.6
	21000	7.1	4.8	11.9	2.9
	Average	**7.0**	**6.9**	**13.9**	**4.9**
Section 3	22000	7.1	2.7	9.8	0.8
	23000	7.1	3	10.1	1.1
	24000	7.1	2.9	10	1
	25000	7.1	2.7	9.8	0.8
	26000	7.1	2.6	9.7	0.7
	27000	7.1	2.4	9.5	0.5
	28000	7.1	1.4	8.5	-0.5
	Average	**7.1**	**2.5**	**9.6**	**0.6**
Section 4	29000	7	0	7	-2
	30000	7	0	7	-2
	31000	7	0	7	-2
	32000	7	0	7	-2
	33000	7	0	7	-2
	34000	7	0	7	-2
	Average	**7.0**	**0.0**	**7.0**	**-2.0**
Section 5	35000	7	3.9	10.9	1.9
	36000	7	3.8	10.8	1.8
	37000	7	3.9	10.9	1.9
	38000	6.9	4	10.9	1.9
	39000	6.9	4	10.9	1.9

TABLE 4A-1
Flood Overtopping Depth at Webb Track during 100-year Event

Levee Section	Station Location	100-Year Flood Stage[1] (ft)	Wind-Wave Runup[1] (ft)	Maximum Flood Elevation (feet)	Flood Overtopping Depth[2] (ft)
	40000	6.9	3.6	10.5	1.5
	41000	6.9	3.8	10.7	1.7
	42000	6.9	3.7	10.6	1.6
	43000	6.9	3.6	10.5	1.5
	44000	6.8	4	10.8	1.8
	45000	6.8	4	10.8	1.8
	46000	6.8	3.3	10.1	1.1
	47000	6.8	2.9	9.7	0.7
	48000	6.8	3	9.8	0.8
	49000	6.8	3.6	10.4	1.4
	50000	6.7	3.7	10.4	1.4
	51000	6.7	3.7	10.4	1.4
	52000	6.7	3.6	10.3	1.3
	53000	6.7	3.4	10.1	1.1
	54000	6.7	3	9.7	0.7
	55000	6.6	3	9.6	0.6
	56000	6.6	3.2	9.8	0.8
	57000	6.6	3.5	10.1	1.1
	58000	6.6	4.1	10.7	1.7
	Average	**6.8**	**3.6**	**10.4**	**1.4**
Section 6	59000	6.6	2.3	8.9	-0.1
	60000	6.6	2.4	9	0
	61000	6.6	1.8	8.4	-0.6
	62000	6.6	1.6	8.2	-0.8
	63000	6.7	1.4	8.1	-0.9
	64000	6.7	1.5	8.2	-0.8
	65000	6.7	1.5	8.2	-0.8
	66000	6.7	1.5	8.2	-0.8
	67000	6.7	1.6	8.3	-0.7
	68000	6.7	1.6	8.3	-0.7
	Average	**6.7**	**1.7**	**8.4**	**-0.6**

(1). Levee Rehabilitation Study by CALFED (August, 1998)
(2). Levee Crest Elevation is 9.0 feet (DW,)